Evaluation of Three Polymer Modified Asphalt Concretes

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In 1989, the Oregon Department of Transportation initiated a research study to evaluate the field performance of three polymer modified asphalts. The polymer modified asphalts evaluated included (*a*) Styrelf, a polymerized binder with a thermoplastic styrene-butadiene block copolymer (*b*) AC–20R, a polymerized binder with a thermosetting styrene-butadiene latex anionic polymer, and (*C*) CA(P)–1, a polymerized binder with a thermoplastic ethylene-vinyl-acetate random copolymer. The three polymer modified asphalt concretes were constructed in five separate test sections adjacent to each other. In addition to the use of polymer modified asphalt, two control sections with a conventional AC–20 asphalt were also constructed for comparison of the performance. A comprehensive evaluation of the materials used on this project and their performance up to June 1993 is presented. Various laboratory tests on binders and mixtures were performed and the results are discussed. Field surveys were conducted annually, and survey results indicate that primary surface distress on all sections is transverse cracking with varying spacing. The level of severity ranged from low to medium. The AC–20 (control) sections showed a more noticeable loss of aggregate than the polymer modified AC sections. In general, both the AC–20 (control) sections and polymer modified (test) sections have been performing well. There is no clear distinction as to which section is superior. As of today, all sections of pavement have carried over 1.5 million equivalent axle loadings.

The use of additives to improve the performance of asphalt cement and asphalt concrete mixtures has increased in recent years (*1*). Polymer additives to asphalt materials are being advocated as having high potential for improving long-term pavement performance through their ability to enhance the properties of the asphalt binder and of the resulting asphalt concrete mix (*2*). Advantages of polymer additives to asphalt include improved adhesion and cohesion, temperature susceptibility, modulus, resistance to fatigue, resistance to rutting, and durability (*3*).

In 1989, the Oregon Department of Transportation (ODOT) initiated a research study to evaluate the field performance of three polymer modified asphalts (*4*). The polymer modified asphalts evaluated included

- Styrelf, a polymerized binder that met Elf Acquitan's PAC–20 specifications. The additive was a thermoplastic styrene-butadiene block copolymer (SB). Asphalt from Montana was used as a base stock, and the polymer content was 3 percent of the binder weight.
- AC–20R, a polymerized binder that met Asphalt Supply and Service's AC–20R specifications. The additive was a thermosetting styrene-butadiene latex anionic polymer (SBR). The base stock was penetration graded asphalt from Montana, and the polymer content was 2 percent of the binder volume.
- CA(P)–1, a polymerized binder that met Chevron's CA(P)–1 specifications. The additive was Elvax 150W, a thermoplastic ethylene-vinyl-acetate random copolymer produced by DuPont Company. The polymer content was 3 percent of the binder weight.

In addition to the use of polymer modified asphalt, two control sections with a conventional AC–20 asphalt were also constructed for comparison of the performance.

This paper presents a comprehensive evaluation of the materials used on this project and their performance as of June 1993.

PROJECT DESCRIPTION

Project Location

This project is located on the Dalles-California Highway (U.S. Route 97) south of the city of Bend, Oregon, between mile point 141.5 and 150.8. The project is a region with severe climate: cold winters, hot summers, frequent freeze-thaw cycles, frequent snowfalls in the winter, and dramatic temperature swings daily. In the winter, the average daily low temperature is about −6°C (21°F) in January. In the summer, the average daily high temperature is about 28°C (82°F) in July. An average of 11 days with highs over 32°C (90°F) occur annually. There are over 150 freeze-thaw cycles annually. The area also receives an annual average of 305 mm (12 in.) of rain and 991 mm (39 in.) of snow. Daily temperature variations are typically in a range of 17°C (30°F) to 22°C (40°F). In August, a daily temperature swing of 31°C (56°F) has been recorded in the past.

Project Layout

The project consists of seven sections. Sections 1 and 3 were paved with Styrelf mixture. Sections 2 and 5 were paved with AC–20 mixture and are the control sections. Sections 4 and 7 were paved with AC–20R mixture. Section 6 was paved with CA(P)–1 mixture.

The wearing course is 51 mm (2 in.) of ODOT Class F open-graded asphalt concrete (*5*), with or without the aforementioned polymer additives. The base course is 51 mm of ODOT Class B dense-graded asphalt concrete (*5*) with conventional AR–4000 grade asphalt. Both mixes have a maximum stone size of 19 mm (¾ in.). Typical tests such as bulk specific gravity, absorption, soundness, and degradation were performed on the aggregate material, and all results were within the limits of the specifications.

Before construction, the existing pavement had two 3.7-m (12-ft) travel lanes and consisted of varying thicknesses of asphalt concrete (AC), oil mat, and cinder base. To accommodate traffic needs, some

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locations were widened to four travel lanes. The widened sections were constructed with 356 mm (14 in.) of cement treated base and then paved with 51 mm of AC base course and 51 mm of AC wearing course, with or without the polymer additives. The subgrade is powdered pumice soil, basalt boulders, and volcanic cinders.

Condition of the Existing Pavement

Before construction of new AC materials, an extensive survey was conducted to evaluate the type and extent of distress along the existing pavement. Within each section, a 76.2-m long segment that represented conditions of the entire section was selected. For each segment, distress type and severity, including a map of all cracks, was recorded. There was considerable alligator and thermal transverse cracking in all sections. The overall condition rating for the project was poor.

Traffic

Traffic data provided by ODOT indicate that the equivalent axle loading (EAL) in 1988 was approximately 314,000. In the last few years (since the construction of the overlay), the pavements have carried over 1.5 million EALs.

PRECONSTRUCTION ENGINEERING

Binder Properties

Laboratory tests on both original and residue asphalt were performed in 1989. The tests included penetration, viscosity, ring and ball softening point, Fraass point, ductility and elastic recovery, force ductility, and toughness and tenacity.

Table 1 presents a summary of the test results. The consistency tests followed ASTM standard testing procedures. The ductility test was used to measure "extension" properties of the binders and was also used in this project to determine elastic recovery property of the binders. Force ductility is a nonstandard test and is a modification of the conventional ductility test. The test has been described as a means to measure tensile load-deformation characteristics of asphalt and asphalt-rubber binders (*6, 7*). The toughness and tenacity test was also used to measure tensile strength of the binders. The Fraass test was used to assess the cold temperature flexibility of an asphalt.

Consistency Tests

Figure 1 shows the consistency test results for original asphalt and residue asphalt on a single diagram, which has been used by ODOT as a means of assessing temperature susceptibility of a binder. The binder with a steeper viscosity-temperature slope is predicted to be more temperature susceptible than a binder with a flatter viscosity-temperature slope. In Figure 1, it is apparent that the AC–20 asphalt binder has a steeper slope than those polymer modified binders. The same trend may be seen for residue asphalt.

Temperature susceptibility of a binder may also be evaluated in terms of Penetration Index (PI) and Penetration Viscosity Number (PVN). PI is calculated by the following equation (*8*):

$$PI = \frac{30}{1 + 90 \times PTS} - 10$$

where PTS is penetration-temperature-susceptibility and is expressed as follows:

$$PTS = \frac{\log 800 - \log Pen_{25}}{T_{RB} - T_{pen25}}$$

where

Pen_{25} = penetration at 25°C (77°F),
T_{RB} = softening point, and
T_{pen25} = 25°C.

The above relationship indicates that an increase in the PI number is an indication of decrease in temperature susceptibility of a binder.

PVN is another way to evaluate the temperature susceptibility of a binder. It is expressed by the following Equation (*9*):

$$PVN = \frac{4.285 - .7967 (\log P) - \log V}{.7591 - .1858 (\log P)} \times (-1.5)$$

where P is the penetration at 25°C and V is Kinematic viscosity at 135°C (275°F). A high value of PVN indicates a binder that has a low-temperature susceptibility.

The calculated PI and PVN values are presented in Figure 2. The AC–20 binder has the lowest PI and PVN, indicating this binder is more temperature susceptible than polymer modified binders.

Ductility and Elastic Recovery Tests

Ductilities were determined in accordance with ASTM D111 testing procedures. At 4°C (39.2°F), ductilities for polymer modified binders were higher than the conventional AC–20 asphalt. There was no noticeable difference for ductility tested at 25°C.

Elastic recovery test results indicate similar characteristics; binders with polymer additives had considerably higher elastic recovery than conventional AC–20 binder. Among the polymer modified binders, Styrelf and AC–20R appeared more ductile and had higher elastic recovery than CA(P)–1.

Force Ductility Tests

Force ductility tests were conducted in accordance with the conventional ductility test with several changes. Two force cells were added to the loading chain and the mold was modified to produce a specimen with constant cross-sectional area through the gage length. The force ductility test data were used to determine the maximum engineering stress, engineering strain, and engineering work. A majority of researchers seem to believe that this test is a significant binder test and an improvement over the conventional ductility test (*2*.) However, other findings (*10*) reported that force ductility test results did not correlate well with low-temperature creep or with fatigue test results for the binder-aggregate mixture.

The force ductility tests were conducted at 4°C and 25°C. The maximum engineering stress was obtained by dividing the maximum load by the original cross-sectioned area. Maximum engineering strain was calculated by dividing the length at failure of the

TABLE 1 Binder Properties (Original and Residue)

Test	AC-20		Styrelf		AC-20R		CA(P)-1	
	Original	Residue	Original	Residue	Original	Residue	Original	Residue
Pen @ 39.2°F, 200g, 60s (dmm)	30	23	50	37	48	34	43	25
Pen @ 77°F, 100g, 5s (dmm)	70 50 min	37	111 60 min	67	105	58	112 85 min	50
Abs Vis @ 140°F (p)	2150 1600-2400	6420	2008 1600-2400	5675	1890 1600-2400	5058	1857 1600-2400	5000
Kin Vis @ 275°F (cSt)	395 230 min	665	572 300 min	960	649 325 min	877	607 325 min	1100
R&B Softening Point (°F)	125	144	130	140	133	140	124	142
Ductility @ 39.2°F 5cm/min (cm)	7	0	47	24	50+ 50 min	45+	42+ 25 min	22
Ductility @ 77°F 5cm/min (cm)	150+	100+	150+	54	100+ 100 min	100+	100+ 100 min	100+
Elastic Recovery @ 50°F (%)	10	-	68 58 min	68	58 58 min	53	35	35
Force-Ductility @ 39.2°F								
Max. Engr. Stress (lb/in^2)	100	220	34	77	49	77	51	100
Max. Engr. Strain (in/in)	33+	6.5	33	19	47+	26	21	15
Max. Engr. Work (lb-in)	54	92	81	130	130	130	130	150
Force-Ductility @ 77°F								
Max. Engr. Stress (lb/in^2)	0.8	4.2	0.4	1.3	1.1	1.5	0.2	2.9
Max. Engr. Strain (in/in)	47+	47+	47+	47+	47+	47+	47+	47+
Max. Engr. Work (lb-in)	0.5	1.7	1.3	5.1	6.7	1.5	0.2	3.9
Toughness (lb-in)	76	138	174	119	216 100 min	164	165 75 min	196
Tenacity (lb-in)	37	52	152	68	197 75 min	115	141 50 min	147
Fraass Point (°F)	19	23	-1	3	3	4	9	17

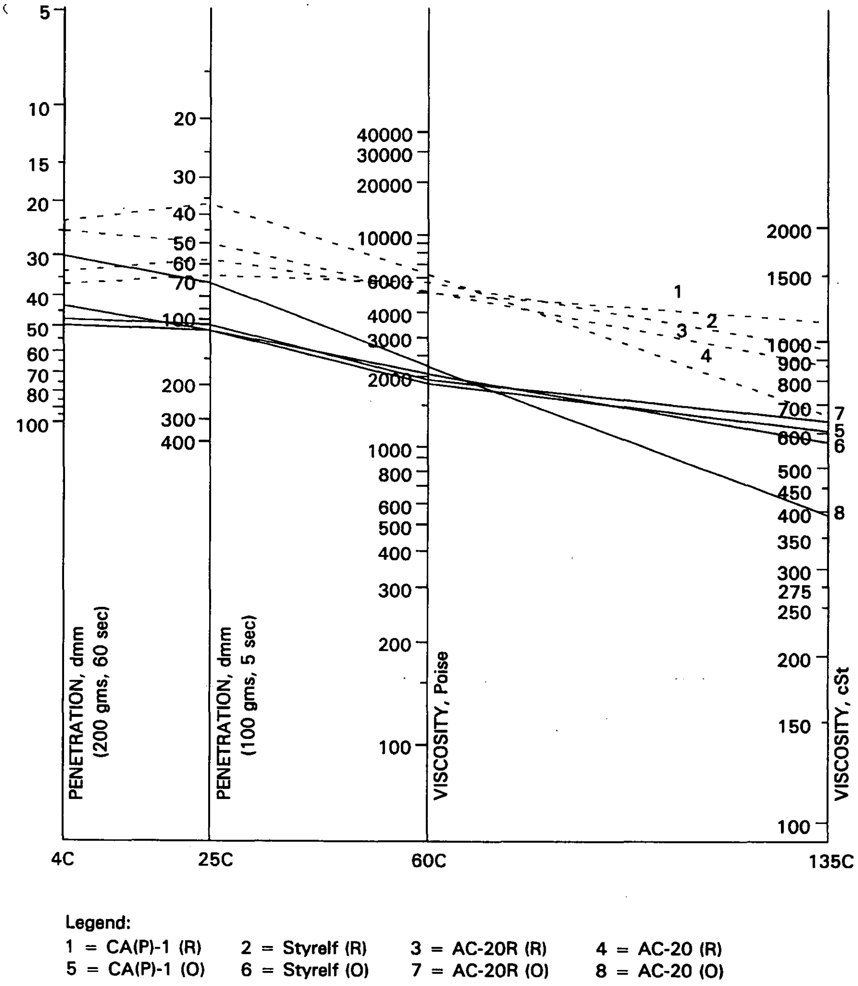

FIGURE 1 **Consistency test results on binders.**

specimen by the original length. Maximum engineering work is the area under the stress-strain curve and could be considered as energy required to produce failure. For binders containing polymer additives, marked increases at 4°C in energy required to produce failure were noticed. This characteristic may be useful in predicting mixture tensile strength when polymer additives are used in binders.

Toughness and Tenacity Tests

The toughness and tenacity tests were performed by placing a tension head into a standard 3-oz penetration tin containing 36 g of binder; the tension head was then pulled at 508mm/min while the force versus extension plot was recorded (*2,11*). The total area under the force-extension curve (typically a bell shaped) was calculated and reported as Toughness. The declining side of the curve was extended to the horizontal axis in a straight line and the area to the right of this line was reported as tenacity. Figure 3 illustrates a comparison of the results of the toughness and tenacity tests. The original binders with polymer additives have higher toughness and tenacity values than the conventional AC–20 asphalt. The polymer modified residue asphalt also exhibits the similar characteristics, except for Styrelf. These test results imply that the binders with polymer additives have a higher tensile strength than the binder without.

Fraass Test Results

The Fraass test measures the cold-temperature flexibility of an asphalt (*2,12*). The test results indicate that all polymer modified binders had lower Fraass points than the conventional AC–20 asphalt, which suggests that the polymer modified asphalts are more flexible at cold temperatures than is the conventional AC–20 binder.

Mix Design

Separate mix designs were made for each binder using the ODOT version of the Hveem method (*13*). The design mix characteristics at design binder content for each mix are listed in Table 2. For open-graded mixes, the mix design criteria are slightly different from those of dense-graded mixes. The criteria used to evaluate the mix properties are binder film thickness, voids, and stability at first compaction, stability at second compaction, and index of retained strength.

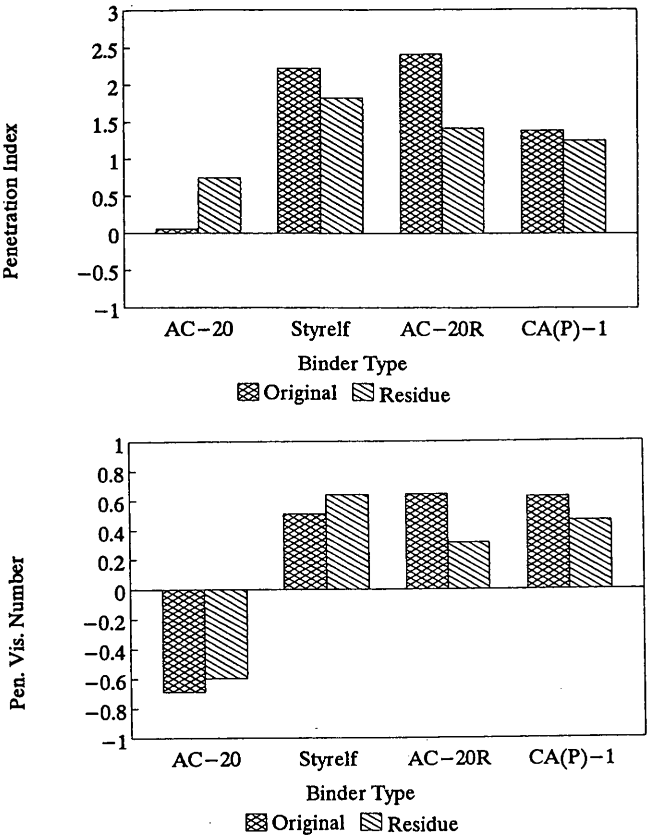

FIGURE 2 Calculations of PI and PVN.

CONSTRUCTION

Placement

The AC–20 mix was used for the construction of the control sections. No unusual problems were reported for this conventional asphalt.

The Styrelf binder mixed easily with the aggregate in the pugmill. Unlike the conventional asphalt, this binder tended to cling to the surfaces of the equipment. Observations indicated that the buildup was moderate and that plant operation was not affected. However, when the binder temperature was at 135°C, the Styrelf binder became very viscous and caused pumping problems that slowed down the batching of the mix production. Consequently, the pumping temperature was raised to range from 149°C (300°F) to 182°C (360°F).

Two other observations on the Styrelf binder were that (a) the binder migrated through the mix to the bottom of the silo when the mix was stored for an extended period, and (b) the binder was especially sensitive to paver speed and screed setting compared with other mixes. For any screed setting, there was a definite maximum paver speed. If the paver exceeded this speed, the screen would rapidly lift. In general, this mix was easy to place.

The AC–20R binder was somewhat similar to Styrelf: easily mixed with the aggregate, tended to cling to the surfaces of the equipment, and was more viscous than AC–20. This binder built up heavier coatings on the surfaces of the equipment than Styrelf, and the buildup was very hard to remove. Migration problems were noticed during construction. In addition, this mix was not smoothly finished by the passage of the screed. There was a minor amount of rolling and picking as the screed passed over the mix. These surface irregularities were not seen after compaction. The mix tended to harden quicker upon cooling than the conventional AC–20, causing difficulty in raking. In general, placement went smoothly and no unusual problems were noticed.

The CA(P)–1 mix is very much like the conventional AC–20 mix except that the smell of fumes from the CA(P)–1 binder were noxious. The placement went easily and otherwise no problems were encountered.

Various quality control tests were conducted during the construction. The test results indicated that the asphalt content and mix gradation were within the specification ranges for all mixes and the average mix placement temperatures generally conformed to or were close to the specifications.

Costs

For a compacted mix 51-mm (2 in.) thick, the Styrelf mix cost $26.75/ton, about 15 percent more than conventional AC–20 mix. Both the AC–20R and CA(P)–1 mixes cost $29.50/ton, about 27 percent more than conventional AC–20 mix. These unit bid prices are for small quantities of binders. The cost would decrease as larger

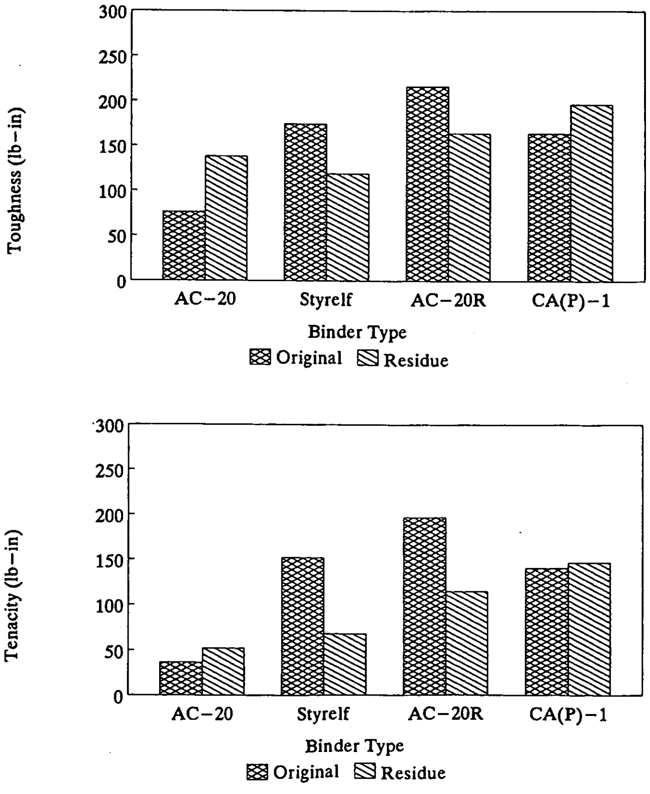

FIGURE 3 Toughness and tenacity test results.

TABLE 2 Design Mix Characteristics at Design Binder Content

Item	AC−20	Styrelf	AC−20R	CA(P)−1	ODOT Class F mix design criteria
Percent passing sieve:					
1"	100	100	100	100	99-100
3/4"	100	98	98	98	95-100
1/2"	76	75	75	75	66-80
3/8"	57	56	56	56	
1/4"	28	25	25	25	18-30
#10	11	10	10	10	5-19
#40	6	6	6	6	
#200	3.6[a]	3.6[a]	3.5[a]	3.5[a]	1.5-6.5[b]
Mineral filler (%)	1[c]	1[c]	1[c]	1[c]	.5-1.5
Voids in mineral agg (%)	19.8	21	21	20	
Binder content (%)	5.2	5.5	5.5	5.5	4-8
Binder film thickness	Suff.	Suff.	Suff.	Suff.	Suff.
SG @ 1st compaction	2.29	2.26	2.27	2.30	
Voids @ 1st comp. (%)	9.0	11.1	9.5	8.7	6-9
Stability @ 1st comp.	24	26	22	21	≥26
SG @ 2nd compaction	2.35	2.36	2.37	2.38	
Voids @ 2nd comp. (%)	6.6	7.2	5.5	5.6	
Stability @ 2nd comp.	37	39	33	32	≥26
Rice Specific Gravity	2.518	2.542	2.509	2.52	
Index of Retained Strength (%)	83	73	84+	76	≥75

[a] Includes loose lime from treated aggregate and 1% fly ash mineral filler.
[b] Includes .5% allowance for loose lime from treated aggregate.
[c] Estimated.

quantities of mix were used, especially if placed by contractors with more experience with the polymer modified AC.

POSTCONSTRUCTION ENGINEERING

Mix Sampling

Mix samples were taken from the discharge chute of the pugmill and sent to the ODOT Materials Laboratory for the determination of asphalt content and aggregate gradation. Some of the observations regarding the mix sampling were that when the mix was hot, the binder would migrate to the bottom of the mix sample; when the mix was cold, it was very difficult to remove all of the binder from the container for testing. In addition, the polymer modified mixes were very tenacious.

Binder Properties

The asphalt in the mixes was extracted to conduct consistency, force ductility, and toughness and tenacity tests. The consistency tests were also performed on recovered asphalt from core samples obtained in 1991, 2 years after construction.

Consistency Tests

The same consistency tests performed on the original binders were conducted on the recovered binders in 1989 and 1991. These tests included penetration and viscosity tests. In addition to tests at 4°C, 200 g, 60 s and at 25°C, 100 g, 5 s, a penetration test was also run at 4°C, 100 g, 5 sec on the binders extracted from core samples obtained in 1991; this test was intended to evaluate the binders' ability to resist low-temperature cracking.

Table 3 summarizes the consistency test results of recovered asphalts for both 1989 and 1991. These results are illustrated in Figure 4 for comparison.

In 2 years the binders' properties had changed considerably: viscosities at 60°C (140°F) changed from approximately 5000 p to over 10000 p; at 135°C (275°F), viscosities also increased—but at different rates, depending on the binders. These changes indicate that the asphalt binders become more viscous after 2 years of road service. The penetration test results also reflect these changes: at

TABLE 3 Binder Properties (Recovered)

Test	AC-20		Styrelf		AC-20R		CA(P)-1	
	1989	1991	1989	1991	1989	1991	1989	1991
Pen @ 39.2°F, 200g, 60s (dmm)	22	17	35	24	26	22	25	14
Pen @ 39.2°F, 100g, 5s (dmm)	N/T	5	N/T	8	N/T	7	N/T	3
Pen @ 77°F, 100g, 5s (dmm)	41	29	63	43	50	35	48	25
Abs Vis @ 140°F (p)	5790	10200	4850	10680	5210	11000	5240	11500
Kin Vis @ 275°F (cSt)	598	811	861	970	964	1040	1090	1380
R&B Softening Point (°F)	144	N/T	140	N/T	136	N/T	140	N/T
Force-Duct @ 39.2°F								
Max. Engr. Stress (lb/in^2)	150	N/T	64	N/T	77	N/T	110	N/T
Max. Engr. Strain (in/in)	7.2		16		19		14	
Max. Engr. Work (lb-in)	64		110		110		160	
Force-Duct @ 77°F								
Max. Engr. Stress (lb/in^2)	2.5	N/T	1.5	N/T	1.6	N/T	2.4	N/T
Max. Engr. Strain (in/in)	47+		47+		47+		47+	
Max. Engr. Work (lb-in)	1.3		7.0		2.1		3.0	
Toughness (lb-in)	80	N/T	87	N/T	124	N/T	219	N/T
Tenacity (lb-in)	18	N/T	48	N/T	56	N/T	101	N/T

N/T = Not Tested

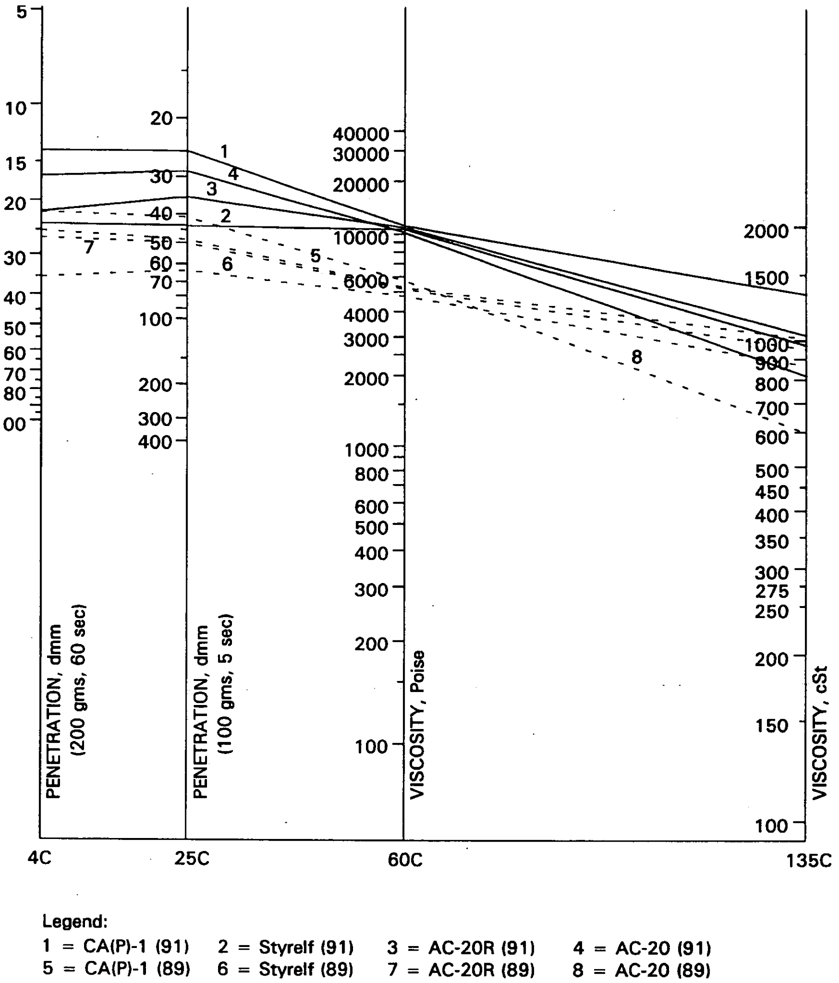

FIGURE 4 Consistency test results on recovered asphalt.

two temperature levels, 4°C and 25°C, the penetration values had decreased nearly proportionally, indicating the binders were much harder in 1991 than they were in 1989.

These test results permitted the reevaluation of temperature susceptibility of the binders. Figure 5 shows a comparison of the calculated PI and PVN for each recovered binder. Based on the PI calculations, the 1989 results indicate the AC–20R binder is the most temperature susceptible. However, the 1991 results indicate the CA(P)–1 binder is the most susceptible to temperature change. PVN calculations present a different picture: calculated PVNs indicate AC–20 has the lowest PVN and is therefore the most susceptible to temperature change.

One additional penetration test at 4°C, 100 g, 5 s was performed in 1991 on binders that were used in the wearing course as well as in the base course. The test was intended to evaluate the binders' ability to resist low-temperature cracking. The test applies a procedure developed by Gaw (*14*) which uses penetration values tested at 4°C and 25°C to predict the temperature at which binder will crack. Based on the Gaw procedure, the predicted cracking temperatures for the binders are

- −40°C (−40°F) for AC–20;
- −45°C (−49°F) for Styrelf;
- −45°C (−49°F) for AC–20R; and
- −30°C (−22°F) for CA(P)–1.

Force Ductility and Toughness and Tenacity Tests

Force ductility and toughness and tenacity tests were conducted on recovered binders in 1989, and the test results are shown in Table 3.

Considerable differences in engineering work at 4°C may be noted: The recovered AC–20 asphalt has the lowest value, meaning that this binder requires the least energy to produce failure. At 25°C, the differences are not significant.

The toughness and tenacity test results show a similar tendency. The AC–20 binder has the lowest toughness and tenacity and the polymer modified binders all have higher toughness and tenacity values, which suggests that they are tougher and more tenacious than conventional AC–20 asphalt.

Mixture Properties

Mixture properties were measured on core samples obtained from the site. The laboratory tests included measurements of bulk specific gravity, Rice specific gravity, Hveem stability, resilient mod-

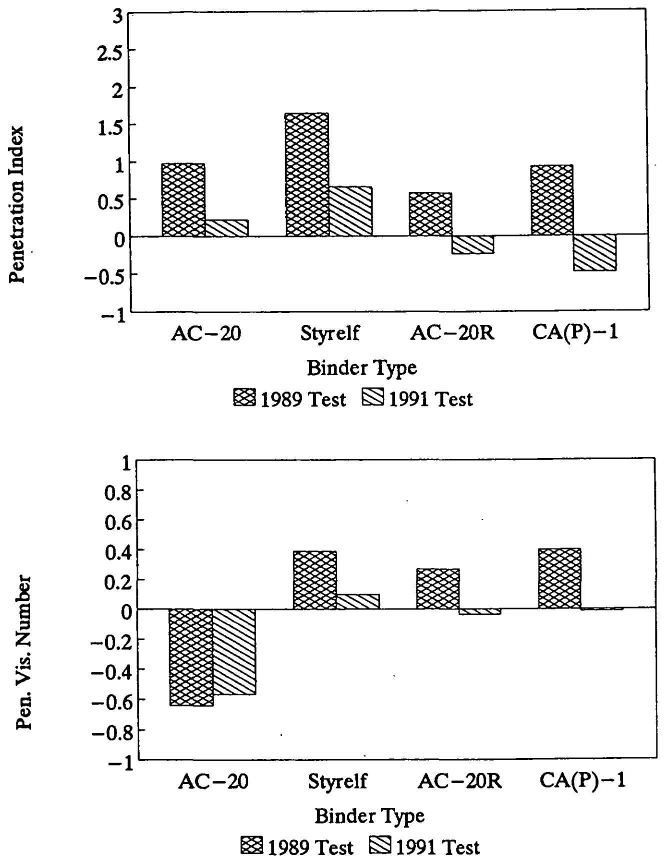

FIGURE 5 Comparison of temperature susceptibility.

ulus, and fatigue life. All tests were performed following ASTM or ODOT standard testing procedures. Table 4 presents the results of various tests.

Bulk and Rice Specific Gravity

The bulk and Rice specific gravity tests were conducted on cores for 3 consecutive years, thus allowing an examination of the change in in-place density and in-place voids of each mixture. Bulk specific gravities of all mixes had been increasing from 1989 to 1990. The rate of change slowed or decreased from 1990 to 1991. As expected, the voids had been decreasing except for the Styrelf and CA(P)–1 mixtures, which show an increase in voids. The determination of voids was based on the test results of both bulk specific gravity and Rice specific gravity, which were determined using recompacted samples. It is possible that samples obtained from two different locations may have slightly different bulk-specific gravities.

Hveem Stability

Hveem stability tests were performed on cores obtained in-place and recompacted mixes and tested in accordance with ODOT Test Method 305-86 (*15*). Two general observations may be concluded from the test results: polymer modified mixes have a similar or slightly higher Hveem stability than the conventional AC–20 mix, and recompacted samples have much higher Hveem stability than in-place cores.

Resilient Modulus

Resilient modulus tests were conducted on cores obtained in-place in 1989, following ASTM D4123 test procedures. The test results indicate that the AC–20R mixture has the lowest resilient modulus; the AC–20 and CA(P)–1 mixtures have a similar resilient modulus; whereas the Styrelf mix has a resilient modulus between those of AC–20 and AC–20R mixtures.

Fatigue

The fatigue tests followed the same procedures as those of resilient modulus tests. The total repetitions of the load to cause failure of the sample was recorded as the fatigue life of the sample. Although these test results may have little meaning to the field performance of each mixture, the AC–20R mixture does show a relatively longer life to resist repeated loading than other mixtures in the controlled laboratory environment. The CA(P)–1 mix, on the other hand, shows a much shorter fatigue life, even shorter than the conventional AC–20 mix.

When the fatigue test results are compared to the resilient modulus test results, it is found that mixes with higher resilient modulus would have lower fatigue life and vice versa.

PERFORMANCE EVALUATION

1989 Survey

Visual Inspection

The visual inspection conducted in 1989 shortly after the construction indicated that the new wearing course of all materials was in an excellent condition. There were no cracks or other types of surface distress. However, it was noticed that before the wearing course was placed, there were 1.6-mm (1/16 in.) to 3.2 mm (1/8 in.) wide transverse cracks completely across the roadway at a frequency of 90 cracks per mile in the new base course, which was constructed directly on the existing pavement in the fall of 1988. There were three possible causes for these cracks: (a) reflective cracking from the existing pavement, (b) shrinkage cracking caused from the cement treated base, and (c) binder's inability of resisting lower temperature cracking. As mentioned earlier, the base course was constructed with AR-4000 asphalt which, from the laboratory test result, had a penetration value of 1 at 4°C, 100 g, 5 s. Based on Gaw's procedure (*14*), the binder with this penetration value would crack at a temperature below −15°C (5°F) by extrapolation. Temperatures in the project area are generally below −18°C (0°F).

Regardless of what caused the cracks in the base course, they could have a considerable effect on the performance of the wearing course.

Friction

The pavement surface friction was measured before construction in 1988 and shortly after construction. All testing was done in accordance with AASHTO T242-84 and was performed at speeds near 40 mph in the left wheel path of the outer lane. The test data was adjusted to standard 40-mph friction number (FN_{40}) using correlation equations. The FNs of the various sections after construction were typical of other new open-graded AC pavements.

TABLE 4 Summary of Core Sample Mix Properties

Test	AC–20	Styrelf	AC–20R	CA(P)–1
a) In-Place				
Bulk Specific Gravity	2.12 (89) 2.26 (90) 2.25 (91)	2.16 (89) 2.26 (90) 2.26 (91)	2.17 (89) 2.24 (90) 2.29 (91)	2.17 (89) 2.25 (90) 2.22 (91)
Voids (%)	14.9 (89) 12.0 (90) 10.7 (91)	17.6 (89) 10.5 (90) 11.3 (91)	14.5 (89) 12.3 (90) 9.4 (91)	14.5 (89) 10.9 (90) 11.6 (91)
Hveem Stability	N/A (89) 11 (90) 15 (91)	13 (89) 16 (90) 22 (91)	14 (89) 15 (90) 15 (91)	13 (89) 16 (90) 14 (91)
Resilient Modulus (ksi)	715[1]	613[2]	339[1]	761[2]
Fatigue (repetitions)	7190[1]	18400[2]	32100[1]	3220[2]
b) Recompacted				
Bulk Specific Gravity	2.33 (89) 2.31 (90) 2.39 (91)	2.29 (89) 2.38 (90) 2.37 (91)	2.30 (89) 2.36 (90) 2.38 (91)	2.29 (89) 2.39 (90) 2.37 (91)
Rice Specific Gravity	2.492 (89) 2.568 (90) 2.519 (91)	2.540 (89) 2.525 (90) 2.550 (91)	2.549 (89) 2.553 (90) 2.525 (91)	2.537 (89) 2.525 (90) 2.516 (91)
Voids (%)	6.5 (89) 10.0 (90) 5.1 (91)	9.8 (89) 5.8 (90) 6.9 (91)	9.8 (89) 7.6 (90) 5.7 (91)	9.7 (89) 5.3 (90) 5.4 (91)
Hveem Stability	17 (89) 32 (90) 30 (91)	25 (89) 28 (90) 50 (91)	23 (89) 51 (90) 37 (91)	24 (89) 29 (90) 31 (91)

Number in parentheses is year.
N/A = Not Available.
[1] Average of two samples. Tested in 1989.
[2] Average of three samples. Tested in 1989.

Roughness

The pavement roughness was measured using a May's ride meter soon after construction in 1989. For all test sections, the roughness values are in a range of 30 to 56 in./mi. Therefore, all test sections were classified as smooth under the ODOT paving award criteria (*4*).

Surveys Since 1990

Visual Inspection

From 1990 to 1993, four separate pavement condition surveys were conducted. Table 5 summarizes the survey results. General observations from these surveys include the following:

1. In 1990, there were no transverse cracks in Sections 1 to 4. There was an average of 10 cracks per mile in Section 5 (AC–20); 2 cracks per mile in Section 6 (Styrelf); and 2 cracks per mile in Section 7 (AC–20R). There was no rutting in Section 1 (Styrelf), approximately 3.2 mm ($^1/_8$ in.) ruts in section 2 (AC–20), and about 1.6mm ($^1/_{16}$ in.) ruts in all other sections.
2. In 1991, all sections had transverse cracks at varying spacing and with a low level of severity. Section 1 (Styrelf) had more transverse cracks per mile than other sections. Section 4 (AC–20R) had the least amount of transverse cracks per mile. All transverse cracks were low level in severity. Rut depths on all sections were generally the same as measured the previous year. No stripping was observed from visual examinations of the cores.
3. The survey results from 1992 and 1993 indicate that there were no new transverse cracks. Some of the existing ones extended slightly towards the pavement edge. The level of severity of the cracks increased on most of the sections from low to medium, particularly on the AC–20 and CA(P)–1 sections. Rut depths on most of the sections also increased slightly from 1992 to 1993.

Friction

Additional friction tests were performed in the summer of 1991. The FNs were slightly higher for all sections compared to FNs measured in 1989. The FNs are typical of other relatively new AC pavements.

TABLE 5 Summary of Condition Survey Since 1990

Sect.	Mix Type	Average Crack Spacing (ft)	Severity	Rutting (inch)	Remarks
1990					
1	Styrelf	No cracks	N/A	0	All sections were in an excellent condition.
2	AC−20	No cracks	N/A	1/8	
3	Styrelf	No cracks	N/A	1/16	
4	AC−20R	No cracks	N/A	1/16	
5	AC−20	~530	L	1/16	
6	CA(P)−1	~2640	L	1/16	
7	AC−20R	~2640	L	1/16	
1991					
1	Styrelf	~80	L	1/16	Cores were taken from all sections for examination of stripping. No stripping was found.
2	AC−20	~140	L	1/8	
3	Styrelf	~135	L	1/16	
4	AC−20R	~165	L	1/16	
5	AC−20	~115	L	1/16	
6	CA(P)−1	~120	L	1/16	
7	AC−20R	~110	L	1/16	
1992					
1	Styrelf	~80	L	1/6	No new transverse cracks were observed. Some of the existing ones extended towards the pavement edge.
2	AC−20	~140	L	1/4	
3	Styrelf	~135	L	1/8	
4	AC−20R	~165	L	1/8	
5	AC−20	~115	L	1/8	
6	CA(P)−1	~120	L	1/8	
7	AC−20R	~110	L	1/8	
1993					
1	Styrelf	~80	L to M	1/6	No new transverse cracks were observed. There were noticeable losses of aggregate on the AC−20 sections.
2	AC−20	~140	M	1/4	
3	Styrelf	~135	L to M	1/6	
4	AC−20R	~165	L to M	1/6	
5	AC−20	~115	M	1/6	
6	CA(P)−1	~120	M	1/6	
7	AC−20R	~110	L	1/6	

N/A = Not Applicable: L = Low. M = Medium.

Roughness

Roughness tests performed in 1990 indicated that the pavement surfaces are slightly rougher than they were in 1989. The roughness values were in a range of 41 to 55 in./mi for all sections. Percent of increase from 1989 to 1990 varies for each section. The two AC–20 sections showed greatest increase (> 30 percent) in roughness. However, all sections of pavement surface were considered smooth based on ODOT paving award criteria.

Discussion of Results

Comparing the laboratory test results to field performance, it appears that force-ductility, and toughness and tenacity tests may be able to assess the binder's tensile and tenacious properties. In the 1993 survey, the loss of coarse aggregate in the wheel path in the AC–20 sections was more noticeable than the loss from the polymer modified AC sections. This seems to support the force-ductility, and toughness and tenacity test results, which all indicated that AC–20 asphalt had lowest values among all binders.

The Gaw procedure predicted a much lower cracking temperature than those measured in the laboratory Fraass test. Based on the Gaw procedure, all binders should be able to resist low-temperature cracking up to at least −30°C (−22°F). In the last few years, the lowest temperature recorded at the project site was not less than −30°C (−22°F). Therefore, the transverse cracks that are in all sections of the pavement may have resulted from either reflective cracking of the existing pavement or shrinkage cracking of the cement treated base; or the Gaw procedure may have overestimated the temperature at which the binder would crack.

CONCLUSIONS

1. The laboratory test results indicate polymer modified asphalt [Styrelf, AC–20R, and CA(P)–1] could be less temperature susceptible than the conventional AC–20 asphalt.

2. The polymer modified binders are much tougher and more tenacious and ductile than the conventional AC–20 asphalt.

3. Fraass test results show the Styrelf, AC–20R, and CA(P)–1 asphalts have lower cracking temperatures than the conventional AC–20 asphalt.

4. Conventional construction processes are suitable to the construction of the Styrelf, AC–20R, and CA(P)–1 modified asphalt mixtures. The Styrelf and AC–20R asphalt tended to migrate to the bottom of the mix. Therefore, appropriate control of mixing temperature is important.

5. The laboratory test results showed that the conventional AC–20 had a higher resilient modulus than the Styrelf and the AC–20R modified AC, and a similar resilient modulus to that of the CA(P)–1 modified AC. The conventional AC–20 AC had a lower fatigue life than the Styrelf and the AC–20R modified AC but had a slightly higher fatigue life than that of CA(P)–1 modified AC.

6. The primary surface distress in all sections is transverse cracking. The level of severity ranged from low to medium. The AC–20 sections showed a more noticeable loss of aggregate than the polymer modified AC sections.

7. The roughness test results showed the Styrelf sections are slightly rougher than other sections, but the AC–20 sections had the greatest increase in roughness, from 1989 to 1990.

8. In general, both the AC–20 (control) sections and polymer modified (test) sections have been performing well. There is no clear distinction as to which section is superior. As of today, all sections of pavement have carried over 1.5 million EALs.

RECOMMENDATIONS

If funding is available, the performance of both the test and control sections should be monitored periodically until the pavement sections fail. Resilient modulus and fatigue tests on core samples, consistency tests on recovered asphalt from both conventional and polymer modified AC, and the friction and roughness tests should be performed every 2 years. Analysis of the above recommended tests will allow ODOT to determine the cost-effectiveness of the three polymer modified asphalt concrete pavements.

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REFERENCES

1. Carpenter, S. H. and T. VanDam. Laboratory Performance Comparisons of Polymer-Modified and Unmodified Asphalt Concrete Mixtures. In *Transportation Research Record 1115,* TRB, National Research Council, Washington, D.C., 1987, pp. 62–74.
2. Rogge, D. F., C. Ifft, R. G. Hicks, and L. G. Scholl. *Laboratory Study of Test Methods for Polymer Modified Asphalt in Hot Mix Pavement.* Final Report, FHWA-OR-RD-90-06. FHWA, U.S. Department of Transportation, Nov. 1989.
3. Terrel, R. L. and J. L. Walter. "Modified Asphalt Pavement Materials: The European Experience," *Proc., Association of Asphalt Paving Technologists,* Clearwater, Fla., Feb. 1986, pp. 482–518.
4. Miller, B. and L. G. Scholl. *Field Test of Polymer Modified Asphalt Concrete: Murphy Road to Lava Butte Section.* Construction Report FHWA-OR-RD-90-07. FHWA, U.S. Department of Transportation, Dec. 1990.
5. Oregon State Highway Division. *Standard Specifications for Highway Construction.* Salem, Oreg., 1991.
6. Salam, Y. *Characterization of Deformation and Fracture of Asphalt Concrete.* Institute of Transportation and Traffic Engineering Series No. 1971:1. University of California, Berkeley, Jan. 1971.
7. Way, B. "Prevention of Reflective Cracking Minnetonka-East," Appendix B (1979 Addendum Report) August 1979.
8. Pfeiffer, *The Properties of Asphalt Bitumen.* Elsevier Science Publishing Company, New York, 1950.
9. McLeod, N.W. Asphalt Cements: Pen-Vis Number and Its Application to Moduli of Stiffness. *ASTM Journal of Testing and Evaluation,* Vol. 4, No. 4, July 1976.
10. Goodrich, J.L. Asphalt and Polymer Modified Asphalt Properties Related to the Performance of Asphalt Concrete Mixes. *Proc., Association of Asphalt Paving Technologists,* Williamsburg, Va., Feb. 1988.
11. Reinke, G. and T. O'Connell. Use of the Toughness and Tenacity Test in the Analysis of Polymer Modified Binders. *Proc., Asphalt Emulsion Manufacturers Association,* New Orleans, La. March 1985, 25 pp.
12. Thenoux, G., G. Lees, and C. Bell. Laboratory Investigation of the Fraass Brittle Test. *Proc., Association of Asphalt Paving Technologists,* San Antonio, Tex., 1985, pp. 529–550.
13. Quinn, W.J. et al., Mix Design Procedures and Guidelines for Asphalt Concrete, Cement Treated Base, and Portland Cement Concrete. Oregon Department of Transportation, Salem, Oreg., April 1987.
14. Gaw, W.J. The Measurement and Prediction of Asphalt Stiffness at Low and Intermediate Pavement Service Temperatures. *Proc., Association of Asphalt Paving Technologists,* 1978.
15. Oregon State Highway Division. *Laboratory Manual of Test Procedures.* Oregon Department of Transportation, Salem, Oreg., 1986.

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