Evaluation of Low-Temperature Pavement Cracking on Elk County Research Project

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The Elk County Research Project in Pennsylvania consists of six test pavements constructed in September 1976 by using AC-20 asphalt cements from different sources. Two test pavements developed extensive low-temperature non-load-associated cracking during the first severe winter. After 2.5 years in service and two more severe winters, the remaining four test pavements do not exhibit any significant cracking. Periodic performance evaluation of these pavements has been conducted. Cracking of the two test pavements was attributed to high stiffness moduli of asphalt cement and asphaltic concrete, determined by indirect nomograph methods. It was felt that the stiffness moduli should also be determined by direct measurements on actual pavement cores. Split tensile tests were conducted at four below-freezing temperatures at a deformation rate of 1.27 mm/s (0.05 in/min) to determine such basic mix properties as stiffness modulus, tensile strength, and tensile strain at failure. Stiffness moduli of the aged pavements were also determined by the indirect Heukelom and McLeod methods. The data from the tensile test indicate that tensile strength or tensile strain at failure, considered independently, does not explain the low-temperature cracking phenomenon on this project. Both direct measurements and indirect methods show that the stiffness modulus of the asphaltic concrete is a better indicator of potential low-temperature cracking. Stiffness moduli determined from the tensile test were generally found to be lower than those obtained by the two indirect methods. A maximum permissible stiffness modulus of 26.9 MPa (3900 lbf/in^2) for original asphalt cement (at minimum pavement design temperature and 20 000 s loading time) has been selected to develop AC-20 asphalt cement specifications for the cold regions of Pennsylvania.

The Elk County Research Project in Pennsylvania consists of six test pavements constructed in September 1976 by using AC-20 asphalt cements from different sources. This research was undertaken with the cooperation of the Federal Highway Administration (FHWA) of the U.S. Department of Transportation to study the low-temperature properties of asphalt cements and asphaltic concrete and their effect on pavement performance and durability.

The winter following the construction of these test pavements (1976-1977) was very severe in Pennsylvania. Visual observation of the six test pavements after the winter revealed that two test pavements had developed extensive low-temperature-associated cracking. The air temperature at the nearest weather station (Ridgway) was as low as -29°C (-20°F) during that winter.

The cracking of these two pavements was attributed to the high stiffness moduli of asphalt cements and asphaltic concrete determined by three indirect methods that use nomographs: the van der Poel, the Heukelom, and the McLeod methods (1).

Subsequently, a laboratory evaluation of Marshall specimens that incorporated these six asphalt cements was made to determine the basic mix properties, such as stiffness modulus, tensile strength, and tensile strain at failure, by using the indirect tensile test. A temperature range of 4-60°C (39.2-140°F) and a relatively high deformation rate of 0.84 mm/s (2 in/min) were used. The measurements indicated (2) that the mixtures in two cracked test pavements had much higher stiffness moduli values than the other test pavements.

It was felt that actual pavement cores should be tested in the lower temperature range of -29 to -12.2°C (-20 to 10°F) and at a lower rate of loading of 1.27 mm/min (0.05 in/min) to evaluate the low-temperature pavement cracking. It has been attempted in this study.

DETAILS OF TEST PAVEMENTS

The project is located in Elk County (north-central Pennsylvania) on US-219 just north of Wilcox. The average daily traffic (ADT) on this two-lane, 6.1-m (20-ft) wide highway is 3700. The research project consisted of 38-mm (1.5-in) resurfacing of the existing structurally sound pavement so that the performance of each test pavement can be studied on a comparative basis. The pavement has been built as follows: 254-mm (10-in) crushed aggregate base and 76-mm (3-in) penetration macadam (1948), 76-mm (3-in) binder and 25-mm (1-in) coarse sand mix (1962), surface treatment (1974), and 38-mm (1.5-in) bituminous concrete wearing course (1976). The subgrade consists of silty soil of AASHTO classification A-4.

The layout plan of the six test pavements and construction details are given elsewhere (1). Each test pavement is approximately 610 m (2000 ft) long. The mix composition and compaction levels were held reasonably consistent on all test pavements. The only significant variable is the asphalt type or source.

The mix consisted of gravel coarse aggregate and natural sand. The mix composition data are given below (1 mm = 0.039 in):

Sieve Size (mm)	Percent Passing	Sieve Size (mm)	Percent Passing
12.5	100	0.6	22
9.5	93	0.3	12
4.75	62	0.15	9
2.36	45	0.075	5
1.18	33		

The asphalt content by weight of mix was 7.5 percent. The Marshall design data for the mix were as follows: theoretical maximum specific gravity (ASTM D2041) = 2.326, specimen specific gravity = 2.278, percentage of voids in mineral aggregate (VMA) = 18.8, percentage of air voids = 2.1, stability = 943 kg (2075 lb), and flow = 3.3 mm (0.13 in). This mix composition has been used in the past and has given durable pavements.

Since the six AC-20 asphalt cements had different viscosities at 135°C (275°F) and 60°C (140°F), the mix temperature for each test pavement was adjusted to obtain a mixing viscosity of 170 ± 20 mm^2/s (170 ± 20 cSt). This helped to obtain an almost consistent compaction level throughout the project.

PROPERTIES OF AC-20 ASPHALT CEMENTS

The AC-20 asphalt cements were supplied by five refineries. Asphalts T-1 and T-5 came from the same refinery. Table 1 gives the sources of crude, methods of refining, and chemical compositions. The properties of original asphalt cements sampled from the tankers at the bituminous concrete plant are given in Table 2, along with data from the thin-film oven test (TFOT) residue. Asphalt was also recovered from the cores taken just after construction; the recovered asphalt properties are

given in Table 3. In May 1978 (20 months after construction), 102-mm (4-in) diameter pavement cores were obtained for this study. After the indirect tensile tests, asphalt was recovered from these cores. The properties of these recovered asphalts are also given in Table 3.

PERFORMANCE OF TEST PAVEMENTS

Weather Data

The Pennsylvania Department of Transportation (PennDOT) has a thermocouple installation site at Lantz Corners, 11 km (7 miles) north of the project, that is capable of recording hourly air temperature and bituminous pavement temperature at 51 mm (2 in) below the surface. According to the recorded data, the critical rapid cooling is believed to have occurred on January 28 and 29, 1977. The air temperature dropped 14°C (25°F) in 2 h. Rapid cooling of pavement 51 mm (2 in) below the surface occurred 12 h later, a drop of 5°C (9°F) in 1 h.

Table 1. Sources of crude oil, methods of refining, and chemical compositions.

Asphalt Type	Crude Oil Source	Method of Refining	Rostler Analysis[a] (%) A	N	A_1	A_2	P	$\frac{A_1+N}{A_2+P}$
T-1	49% Sahara, 21% West Texas, 21% Montana, 9% Kansas	Vacuum distillation and propane deasphalting	8.1	9.0	39.9	30.9	12.1	1.14
T-2	66.7% Texas mid-continent, 33.3% Arabian	Steam distillation	22.4	17.4	24.4	24.4	11.3	1.17
T-3	85% light Arabian, 15% Bachaquero	Vacuum distillation	17.0	23.2	18.8	31.0	10.0	1.02
T-4	75% West Texas sour, 25% Texas and Louisiana sour	Vacuum distillation	19.4	23.1	17.0	27.7	12.8	0.99
T-5	Same as T-1	Same as T-1	15.9	28.7	18.2	27.7	9.4	1.26
T-6	Blend of heavy Venezuelan and Middle East crude	Vacuum distillation	10.4	25.8	19.1	25.3	19.3	1.01

[a]A = asphaltenes; N = nitrogen bases; A_1 = first acidaffins; A_2 = second acidaffins; P = paraffins.

Table 2. Properties of original AC-20 asphalt cements.

Test	Asphalt Type T-1	T-2	T-3	T-4	T-5	T-6
Original Samples						
Penetration at 100 g, 5 s (0.1 mm)						
At 4°C	2.0	7.4	6.2	6.7	3.4	7.5
At 15.6°C	11.2	25.0	24.5	23.0	16.0	29.0
At 25°C	42	64	72	65	54	80
Viscosity						
Absolute, 60°C (Pa·s)	271.0	228.4	176.4	170.5	175.9	198.2
Kinematic, 135°C (mm²/s)	420	402	393	355	356	406
Softening point, ring and ball (°C)	50.6	50.0	48.9	50.0	51.1	49.4
Penetration index (PI)	-2.77	-0.71	-1.51	-1.05	-2.23	-1.29
Penetration-viscosity number (PVN)	-1.04	-0.70	-0.61	-0.86	-1.03	-0.45
TFOT Residue						
Penetration at 100 g, 5 s (0.1 mm)						
At 25°C	26	38	45	38	37	44
Viscosity						
Absolute, 60°C (Pa·s)	550.1	683.5	398.2	469.4	324.8	572.1
Kinematic, 135°C (mm²/s)	563	569	556	527	464	575
Ductility (cm)						
At 4°C, 1 cm/min	3.5	3.5	4.6	5.2	8.6	12.4
At 15.6°C, 5 cm/min	11.6	7.0	95.2	12.8	90.6	33.0

Note: t°C = (t°F -32)/1.8; 1 Pa·s = 10 poises; 1 mm²/s = 1 centistoke.

Table 3. Properties of recovered AC-20 asphalt cements just after construction and after 20 months.

Test	Just After Construction T-1	T-2	T-3	T-4	T-5	T-6	After 20 Months T-1	T-2	T-3	T-4	T-5	T-6
Penetration at 100 g, 5 s (0.1 mm)												
At 4°C	1.5	4.5	4.5	4.0	2.0	5.8	3.0	5.0	5.0	5.7	2.7	5.0
At 15.6°C	7	17	16	13	9	20	7.0	13.0	14.1	13.0	8.0	13.0
At 25°C	24	40	43	34	29	49	19.0	25.2	36.0	31.0	20.6	29.3
Viscosity												
Absolute, 60°C (Pa·s)	552.6	572.9	378.9	382.9	401.9	461.1	847.4	2273.8	557.0	799.8	816.6	1487.1
Kinematic, 135°C (mm²/s)	565	569	526	487	488	576	690	892	625	641	655	890
Softening point, ring and ball (°C)	56.7	53.3	53.9	53.3	54.4	53.9	58.9	64.7	57.1	61.7	59.4	62.2
Ductility (cm)												
At 4°C, 1 cm/min	0.2	4.6	13.9	5.9	0.6	14.9	1.5	1.0	5.0	5.5	4.1	4.8
At 15.6°C, 5 cm/min	8.3	7.2	48.5	10.0	15.5	34.0	1.5	3.4	12.2	4.5	4.4	5.6
At 25°C, 5 cm/min	150+	80	150+	150+	150+	150+	150+	76	150+	47.5	125.1	42.6
Penetration index (PI)	-2.24	-0.80	-0.99	-0.65	-2.03	-0.64	+0.36	+1.22	-0.12	+0.93	-0.32	+0.60
Penetration-viscosity number (PVN)	-1.13	-0.68	-0.72	-1.03	-1.16	-0.47	-1.07	-0.54	-0.65	-0.76	-1.07	-0.4

Note: t°C = (t°F -32)/1.8; 1 Pa·s = 10 poises; 1 mm²/s = 1 centistoke.

Table 4. Survey of transverse cracking in pavements T-1 and T-5.

Pavement Section	Time	No. of Transverse Cracks Full (F)	Half (H)	Part (P)	Cracking Index[a] (500 ft)
T-1, station 205+00 to station 215+00	October 1977	5	41	102	51
	May 1978	6	44	164	69
	May 1979	7	45	184	76
T-5, station 125+00 to station 135+00	October 1977	11	26	58	38
	May 1978	14	28	88	50
	May 1979	21	35	64	54

[a]Cracking index = F + 0.5 H + 0.25 P.

The minimum air temperature recorded was -29°C (-20°F), whereas the pavement temperature 51 mm below the surface reached -23°C (-10°F). The 1976-1977 air freezing index for this site was determined to be 1509 degree-days. As mentioned earlier, test pavements T-1 and T-5 developed excessive low-temperature-associated shrinkage cracking during this winter.

Low ambient temperatures prevailed again at the experimental site during the second (1977-1978) and the third (1978-1979) winters. The following minimum air temperatures were recorded at the nearest U.S. weather station (Ridgway), 22.5 km (14 miles) south of the project [t°C = (t°F - 32)/1.8]:

Date	Minimum Air Temperature (°C)	Date	Minimum Air Temperature (°C)
1/23/78	-25.6	2/12/79	-31.7
1/24/78	-25.6	2/13/79	-27.2
2/04/78	-27.8	2/14/79	-28.9
2/05/78	-27.2	2/15/79	-27.2
1/12/79	-26.1	2/17/79	-30.6
2/10/79	-26.1	2/18/79	-30.6
2/11/79	-31.7		

It is estimated that the pavement temperature 51 mm (2 in) below the surface must have reached -23°C (-10°F), which had been considered the design temperature previously for the cold regions of Pennsylvania (1).

Visual Evaluation

When the six test pavements were constructed in September 1976, no visual differences could be seen among them. These pavements have been evaluated periodically since then by a team of 8-10 evaluators. The last inspection was made in May 1979, after the pavements had undergone three unusually severe winters. The pictures of the pavements in this paper were taken in May 1979. A periodic crack survey (Table 4) indicates that the test pavements T-1 and T-5 are developing more cracks and that the existing cracks appear to widen after each successive winter. Test pavements T-2, T-3, T-4, and T-6 have not developed any significant transverse or longitudinal cracking so far.

Figure 1 shows that asphalt T-1 in both lanes has numerous transverse and longitudinal cracks. Figure 2 shows part of the same pavement that has developed block cracking that has produced deterioration and formation of potholes at some places. Figure 3 illustrates the classic effect of C_v (the factor for volume concentration of aggregate) on the mix stiffness modulus. Both lanes have the same asphalt T-1, but the background lane was compacted to 4.4 percent average voids, whereas the foreground lane has approximately 9 percent voids (the result of an isolated cold-mix load). It can be seen that the transverse cracks are mostly confined to the background lane. The mix in the foreground lane has a lower stiffness modulus because of higher air voids and is thus less susceptible to low-temperature cracking. However, higher air voids have induced surface raveling in this area. It appears that the asphaltic mixtures designed with a high VMA by adjusting gradation should also have low stiffness moduli values and thus would help to minimize the cracking.

Figure 4 shows asphalt T-1, which has block cracking, in the foreground lane and asphalt T-2, which has no cracks, in the adjacent background lane. Figures 5 and 6 show asphalts 3 and 4, respectively, which have no transverse or longitudinal cracking.

Figure 7 shows asphalt T-5 in both lanes; it has numerous full- and half-width transverse cracks. Transverse cracks are better defined in asphalt T-5 than in asphalt T-1. Figure 8 shows cracked asphalt T-5 in the foreground lane and crack-free asphalt T-6 in the adjacent background lane.

TEST PROCEDURE AND CALCULATIONS

The indirect tensile test involves loading a cylindrical specimen with compressive loads that act parallel to and along the vertical diametrical plane. Pavement core specimens 101.6 mm (4 in) in diameter were tested in this study. The average thickness of the cores was 25.4 mm (1 in). To distribute the load and maintain a constant loading area, the compressive load was applied through a 12.7-mm (0.5-in) wide steel loading strip that was curved at the interface with the specimen and had a radius equal to that of the specimen.

This loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametrical plane that ultimately causes the specimen to fail by splitting or rupturing along the vertical diameter. By measuring the applied load at failure and by continuously monitoring the loads and the horizontal and vertical deformations of the specimens, one can estimate the mix tensile strength (S_T), Poisson ratio (ν), and stiffness modulus (S_F). The equipment and test procedure are described elsewhere (3). A deformation rate of 1.27 mm/s (0.05 in/min) was employed.

The theoretical relationships used in calculating S_T, ν, and S_F are complex and require integration of various mathematical functions. However, by assuming a specimen diameter, one can make the required integrations and simplify the relationships (3,4). These simplified relationships for calculating S_T, ν, S_F, and total tensile strain at failure (ϵ_T) for a 101.6-mm (4-in) diameter specimen with a 12.7-mm (0.5-in) wide curved loading strip are as follows (since the equations are formulated in U.S. customary units of measurement, no SI equivalents are given):

$$S_T = 0.156(P_{fail}/h) \quad (1)$$

$$\nu = (0.0673DR - 0.8954)/(-0.2494DR - 0.0156) \quad (2)$$

$$S_F = (SH/h)(0.9976\nu + 0.2692) \quad (3)$$

$$\epsilon_T = X_{TF}[(0.1185\nu + 0.038\,96)/(0.2494\nu + 0.0673)] \quad (4)$$

where

P_{fail} = total load at failure (lb),
h = height of specimen (in),

DR = deformation ratio (Y_T/X_T) = slope of line of best fit between vertical deformation Y_T and the corresponding horizontal deformation X_T in the linear portion only,
S_H = horizontal tangent modulus (P/X_T) = slope of the line of best fit between load P and X_T in the linear portion only, and
X_{TF} = total horizontal deformation at failure.

The line of best fit was determined by the least-squares method. It was felt that a stiffness value obtained from the linear portion of the stress-strain relation would be more meaningful than the failure stiffness.

INDIRECT TENSILE TEST DATA

Tests were conducted at four temperatures: -29°C,

Figure 1. Asphalt T-1 showing transverse and longitudinal cracks in both lanes.

Figure 2. Asphalt T-1 showing block cracking and deterioration.

Figure 3. Asphalt T-1 showing effect of C_v on the extent of cracking.

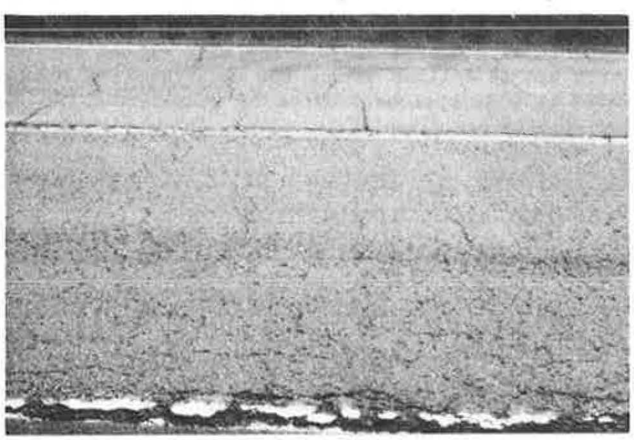


Figure 4. Asphalt T-1 (foreground lane) and asphalt T-2 (background lane).

Figure 5. Asphalt T-3 (both lanes).

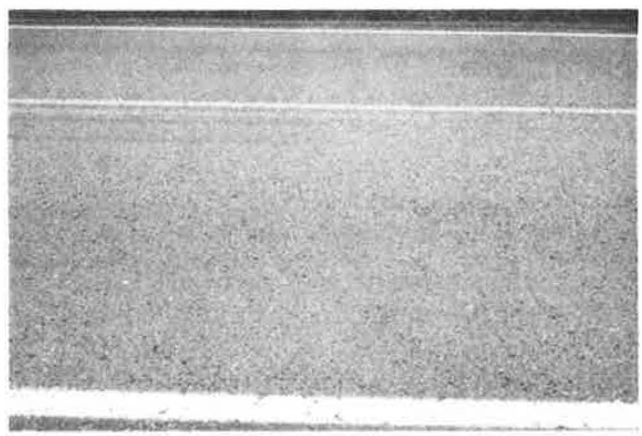


Figure 6. Asphalt T-4 (both lanes).

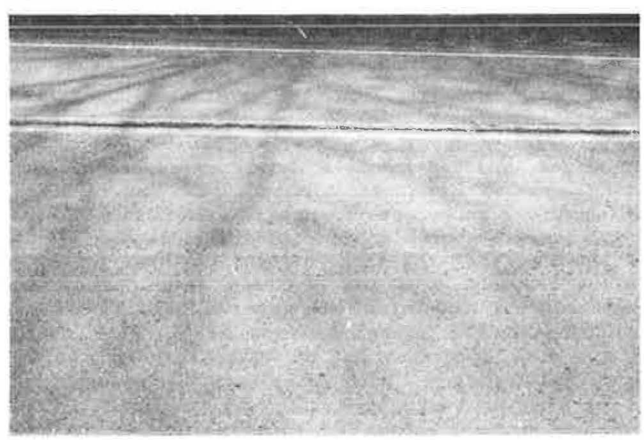


Figure 7. Asphalt T-5 showing well-defined full- and half-width transverse cracks.

Figure 8. Asphalt T-5 (foreground lane) and asphalt T-6 (background lane).

-23.3°C, -17.8°C, and -12.2°C (-20°F, -10°F, 0°F, and 10°F). A minimum of three core specimens from each asphalt type was tested at each temperature, and the results were averaged. Table 5 gives the mix test data on S_T, tensile failure strain (ϵ_F), and S_F for the six asphalts at four temperatures.

Tensile Strength (S_T)

Figure 9 shows the relationship between the temperature and mix S_T by drawing a best-fit curve for each asphalt. In all cases, the S_T of the asphaltic concrete increased as the test temperature was decreased from -12.2 to -29°C (10 to -20°F). The rate of increase of S_T as temperature decreased is lower for asphalts T-1 and T-5 than for the remaining four asphalts. These two asphalts are highly temperature-susceptible, as indicated by their penetration-viscosity numbers (PVNs). Asphalt T-6 has the highest S_T at temperatures of -17.8°C (0°F) and lower; asphalt T-2 has the lowest S_T at all temperatures.

It can be observed that the phenomenon of low-temperature cracking cannot be explained by the mix S_T alone.

Tensile Strain at Failure (ϵ_F)

Figure 10 shows the plot of temperature versus tensile strain at failure (ϵ_F). Asphalts T-1 and T-5 failed at tensile strains lower than those for asphalts T-3, T-4, and T-6 at all temperatures. Some researchers (5) have considered ϵ_F the most significant parameter; the occurrence of cracking was found to increase as ϵ_F decreased. However, asphalt T-2 has ϵ_F comparable to those of asphalts T-1 and T-5 at temperatures of -17.8°C (0°F) and lower, but it has not developed any cracking. Incidentally, asphalt T-2 has much lower S_T at all temperatures (Figure 9) and, therefore, it has a lower failure S_F than asphalts T-1 and T-5. Thus, ϵ_F does not necessarily explain entirely the low-temperature cracking.

Stiffness Modulus (S_F)

Tensile Test

Figure 11 shows the plot of temperature versus failure S_F. Figure 12 shows the plot of temperature versus mix S_F that was obtained from the linear portion of the stress-strain relation, as explained earlier.

Both figures indicate higher S_F's for asphalts T-1 and T-5, which have both developed cracking. Thus, S_F is a better indicator of potential cracking problem than tensile stress or ϵ_F examined independently.

The curves for asphalts T-2, T-3, T-4, and T-6 crisscross each other. Apparently, the differences between their S_F's do not appear to be greater than the reproducibility of the split tensile test conducted on approximately 25.4-mm (1-in) thick pavement cores. Experience has now indicated that at least five cores should be tested so that the outliers can be identified properly. It was observed that the test data on S_T were more consistent than the tensile strain data.

Indirect Methods

Stiffness moduli of the six aged asphalt cements were determined from the data in Table 3 by using two indirect nomograph methods:

1. The Heukelom method (6) uses penetration at two or three temperatures, the "corrected" softening point T_{800}, and the penetration index (PI) (pen/pen). The original van der Poel nomograph is used for determining S_F.
2. The McLeod method (7) uses penetration at 25°C (77°F) and viscosity at 135°C (275°F), base temperature, and PVN.

The S_F of the paving mixture was then determined from the S_F of asphalt cement and $C_V^{\prime}$ (a factor for volume concentration of aggregate) by using the chart developed by van der Poel and given by McLeod (7). Based on the core density data, $C_V^{\prime}$ was determined to be 0.83.

S_F's of the six paving mixtures were determined for 20 000 s loading time at the four temperatures used in the tensile test. The data are plotted in Figures 13 and 14.

Comparison of Direct Measurements and Indirect Determinations

The following observations are made after reviewing Figures 11-14 and Table 3.

1. As expected, failure S_F's (Figure 11) are higher than S_F's obtained from the straight portion of stress-strain curves (Figure 12).
2. Generally, S_F's obtained by the McLeod method are higher than those obtained by the

Table 5. Values of S_T, ϵ_F, and S_F at four temperatures.

	-29°C			-23.3°C			-17.8°C			-12.2°C		
Asphalt Type	S_T (kPa)	ϵ_F (mm/mm $\times 10^{-4}$)	S_F (kPa $\times 10^6$)	S_T (kPa)	ϵ_F (mm/mm $\times 10^{-4}$)	S_F (kPa $\times 10^6$)	S_T (kPa)	ϵ_F (mm/mm $\times 10^{-4}$)	S_F (kPa $\times 10^6$)	S_T (kPa)	ϵ_F (mm/mm $\times 10^4$)	S_F (kPa $\times 10^6$)
T 1	2834	2.94	2.83	2724	3.25	2.18	2648	3.42	1.70	2413	3.62	1.52
T-2	2434	2.95	1.92	2220	3.45	1.22	1903	3.49	1.33	1600	4.36	1.01
T-3	2758	4.07	2.03	2710	3.87	1.54	2537	4.49	1.59	1627	3.99	0.841
T-4	2696	3.64	1.80	2579	3.97	1.59	2331	3.92	1.50	1834	5.14	0.979
T-5	2792	3.27	2.41	2668	3.26	2.01	2599	3.66	1.74	2372	3.75	1.46
T-6	3158	4.40	1.83	2903	4.35	1.90	2751	4.88	1.59	2131	8.42	1.19

Note: $t°C = (t°F - 32)/1.8$; 1 kPa = 0.145 lbf/in².

Figure 9. Temperature versus mix S_T.

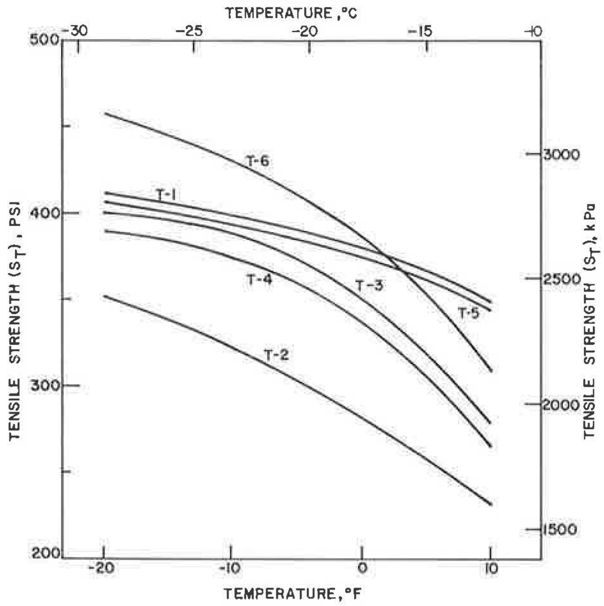


Figure 11. Temperature versus failure S_F.

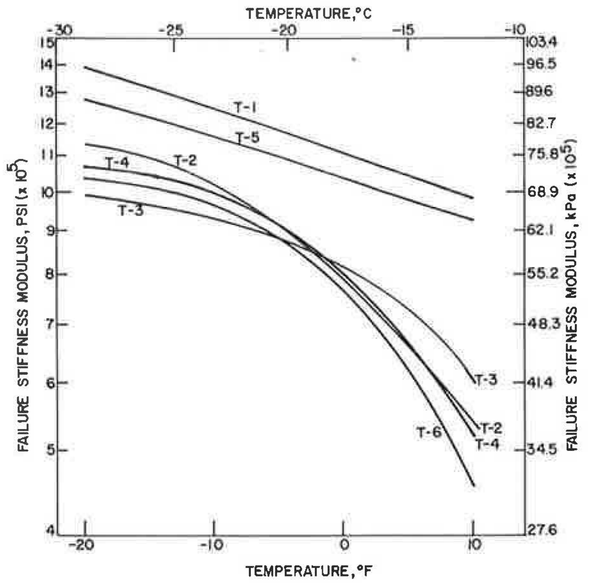


Figure 10. Temperature versus failure ϵ_F.

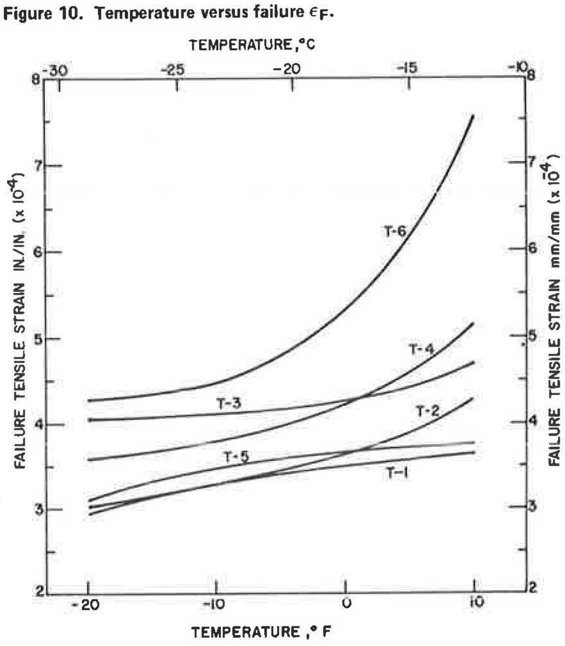


Figure 12. Temperature versus mix S_F.

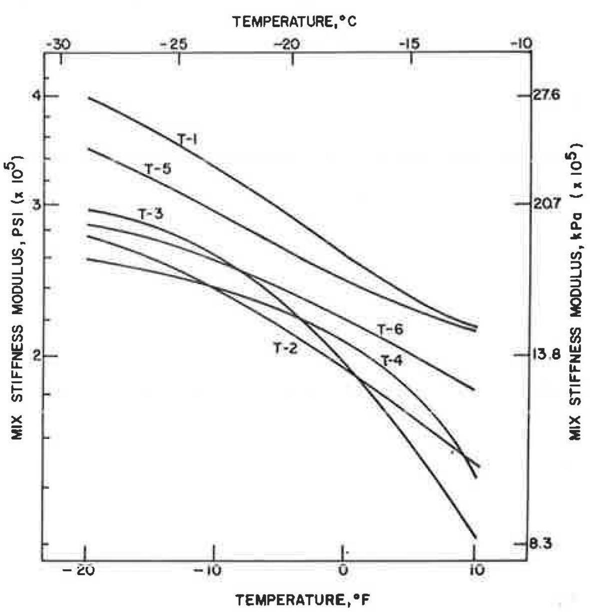


Figure 13. Temperature versus indirect mix S_F (McLeod method).

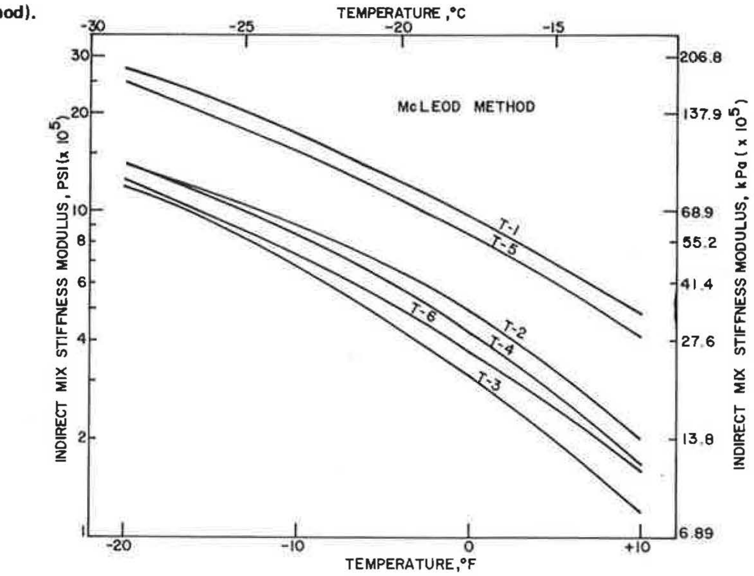
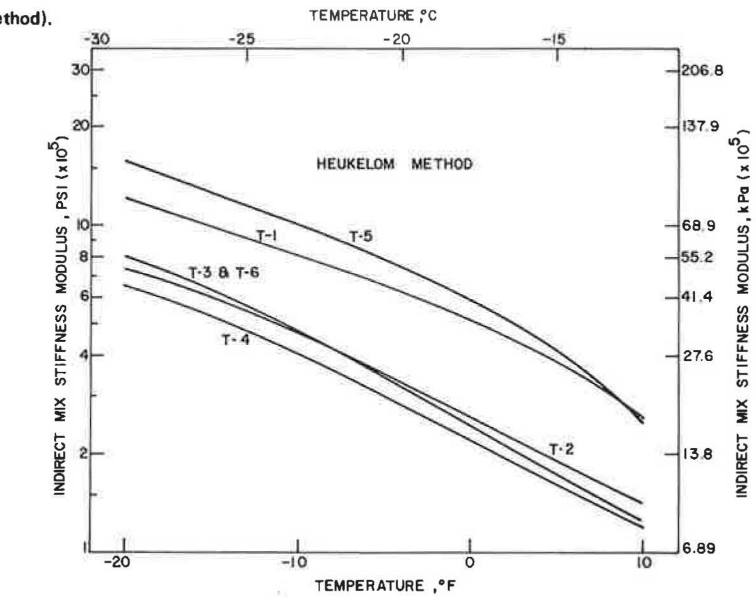


Figure 14. Temperature versus indirect mix S_F (Heukelom method).



Heukelom method, especially at lower temperatures and/or at higher stiffness levels.

3. Since the loading time is different for direct measurements and indirect determinations, it is difficult to make meaningful comparisons. Average loading time to failure in the tensile test was 60 s at a deformation rate of 1.27 mm/s (0.05 in/min). Although a much longer loading time of 20 000 s was used in the indirect determinations, the S_F's obtained are higher than the directly measured values. Generally, the failure S_F values are closer to the indirectly obtained values than are the straight-line S_F's.

Temperature Susceptibility

Temperature susceptibilities of the original and the aged asphalts are reported quantitatively by PI and PVN values in Table 6; these values are used to determine the S_F's of asphalts by the Heukelom and McLeod methods, respectively. It can be observed that PI values have changed drastically after 20 months, whereas PVN values remained essentially unchanged. Because of these drastic changes in the PI values, the S_F's of most asphalts determined by the Heukelom method have decreased rather than increased after 20 months of aging.

Limiting Stiffness Criteria

McLeod (8) has concluded that the low-temperature pavement cracking is likely to become serious if the pavement develops an S_F of 6897 MPa (1×10^6 lbf/in^2) at the lowest pavement temperature to which it is exposed for a loading time of 20 000 s, which corresponds to slow chilling on a cold night. This seems to have been confirmed on this project so far. Pavements T-1 and T-5 have developed stiffness moduli greater than 6897 MPa at -23°C (-10°F) and thus are exhibiting low-temperature cracking. Compared with pavement T-5, pavement T-1 has a higher S_F and thus a higher cracking index (Table 4).

It is difficult to suggest a limiting S_F value that is based on the indirect tensile test data because of the test limitations mentioned earlier.

Table 6. Temperature susceptibilities of original and aged asphalts.

Asphalt Type	PI Original	PI Just After Construction	PI At 20 Months	PVN Original	PVN Just After Construction	PVN At 20 Months
T-1	-2.77	-2.24	+0.36	-1.04	-1.13	-1.07
T-2	-0.71	-0.80	+1.22	-0.70	-0.68	-0.54
T 3	1.51	0.99	0.12	-0.61	-0.72	-0.65
T-4	-1.05	-0.65	+0.93	-0.86	-1.03	-0.76
T-5	-2.23	-2.03	-0.32	-1.03	-1.16	-1.07
T-6	-1.29	-0.64	+0.60	-0.45	-0.47	-0.40

AC-20 Asphalt Cement Specifications

Since cracking similar to that observed in pavements T-1 and T-5 was also observed on several pavements in northwestern Pennsylvania where such asphalt cements were used, it became necessary to develop new specifications for cold regions in Pennsylvania to avoid low-temperature cracking.

It became essential to establish the maximum permissible S_F of the original asphalt. Since the PennDOT specifications specify penetration at 25°C (77°F) and viscosity at 135°C (275°F), it appeared convenient to use the McLeod method for determining the S_F at a design temperature of -23°C (-10°F) and a loading time of 20 000 s. Finn and others (9) have also recommended two designer-controlled approaches to low-temperature cracking: (a) specifications for asphalt cement (penetration and viscosity) and (b) stiffness values of asphalt cement or asphaltic concrete that do not exceed limiting criteria for a particular temperature regime. At the present time, establishing the maximum permissible S_F of the original asphalt seems to be the most workable alternative. The original asphalt cements T-1 and T-5, which developed low-temperature cracking during the first winter, had S_F's of 68.5 MPa (9930 lbf/in²) and 36.7 MPa (5320 lbf/in²), respectively. Asphalt T-4 had the highest S_F--19.6 MPa (2840 lbf/in²)--among the remaining four pavements, which do not exhibit transverse cracking at the present time (2.5 years after construction).

It appeared that the maximum permissible limit for S_F was between 19.6 and 36.7 MPa. More data have to be gathered before such a limit can be fully established. It is quite possible that the other pavements may develop low-temperature cracking after aging in service. However, with the limited data it appeared reasonable to assume 26.9 MPa (3900 lbf/in²), which is midway between 19.6 and 36.7 MPa, as the limiting stiffness criterion. This limit can be lowered or increased later as more data are collected.

By using the limiting stiffness of 26.9 MPa and McLeod's nomograph method (7), minimum allowable PVNs were determined for various penetration values. Minimum kinematic viscosities were then determined from the corresponding penetration and PVN values. By specifying the minimum kinematic viscosity at 135°C (275°F) thus determined for each penetration value at 25°C (77°F), it was ensured that the PVN is not lower than the permissible value. Some values are given below (1 mm = 0.039 in; 1 mm²/s = 1 cSt):

Penetration (0.1 mm)	Minimum PVN Allowable	Specified Minimum Viscosity (mm²/s)
60	-0.80	390
65	-0.95	330
70	-1.10	290
75	-1.25	250

It can be seen that higher-temperature-susceptible asphalts can be used if their penetration values are in accordance with those listed above.

The new AC-20 asphalt cement specifications based on the maximum permissible stiffness criteria were made effective in July 1977 for the colder regions of Pennsylvania. The colder regions are those that have an air freezing index of 1000 degree-days or more based on the 1962-1963 severe winter.

CONCLUSIONS

Based on the data obtained by the direct measurements (split tensile test) and the indirect methods, the following conclusions are drawn.

1. Mix S_T or ε_F does not explain the low-temperature cracking phenomenon entirely, if each is considered independently.
2. Both direct measurements and indirect methods indicate that the S_F of the asphaltic concrete is a better indicator of potential low-temperature cracking. Asphalts T-1 and T-5, which developed such cracking, have higher S_F values than the remaining four asphalts.
3. S_F values obtained by the McLeod method are generally higher than those obtained by the Heukelom method, especially at lower temperatures and/or at higher stiffness levels.
4. Although a much longer loading time of 20 000 s was used in the indirect methods, the S_F values are generally higher than those obtained in the split tensile test that was conducted at a deformation rate of 1.27 mm/s (0.05 in/min) and had an average time to failure of 60 s.
5. Temperature susceptibility of the asphalts as indicated by PI values has changed drastically after 20 months of aging. However, the same property expressed by PVN values is essentially unchanged.
6. Limiting asphaltic concrete S_F criteria of 6897 MPa (1×10^6 lbf/in²) at the lowest pavement temperature for a loading time of 20 000 s to avoid low-temperature cracking has been verified on this research project so far through three unusually severe winters in succession.
7. From the available data, a maximum permissible S_F of 26.9 MPa (3900 lbf/in²) for original asphalt cement (at minimum pavement design temperature and 20 000 s loading time) was selected to develop AC-20 asphalt cement specifications for cold regions of Pennsylvania. This limit can be lowered or increased later as more data are collected.

The evaluation of these six test pavements will be continued to monitor the effect of increasing S_F of asphalt cements resulting from aging in service on the development of low-temperature non-load-associated shrinkage cracking and other forms of pavement distress.

ACKNOWLEDGMENT

This research project was undertaken by PennDOT with the cooperation of FHWA. The opinions, findings, and conclusions expressed here are mine and not necessarily those of PennDOT or FHWA.

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Discussion

H.E. Schweyer, B.E. Ruth, and C.F. Potts

We believe that this work will turn out to be a classic study in the history of asphalt pavement cracking. The work is most important to our programs in Florida in providing field confirmation of some ideas we have been investigating at the University of Florida with the help of the Florida Department of Transportation (FDOT). Kandhal has provided the opportunity for us to demonstrate that our low-temperature rheological research has been worthwhile. He sent us the bitumens and loose-mix samples from each road section. We are studying rheology data of these down to -10°C (14°F); FDOT is assisting in testing the mixtures. These test data will be correlated with the field-service evaluation to confirm a rational rheological explanation that separates different asphalts in service based on temperature susceptibility and non-Newtonian flow rheology. We expect to present a full report later.

Briefly, (a) we have confirmed the higher temperature susceptibility of T-1 and T-5 bitumens and have verified that both samples have higher

Figure 15. Extended scales for temperature susceptibility.

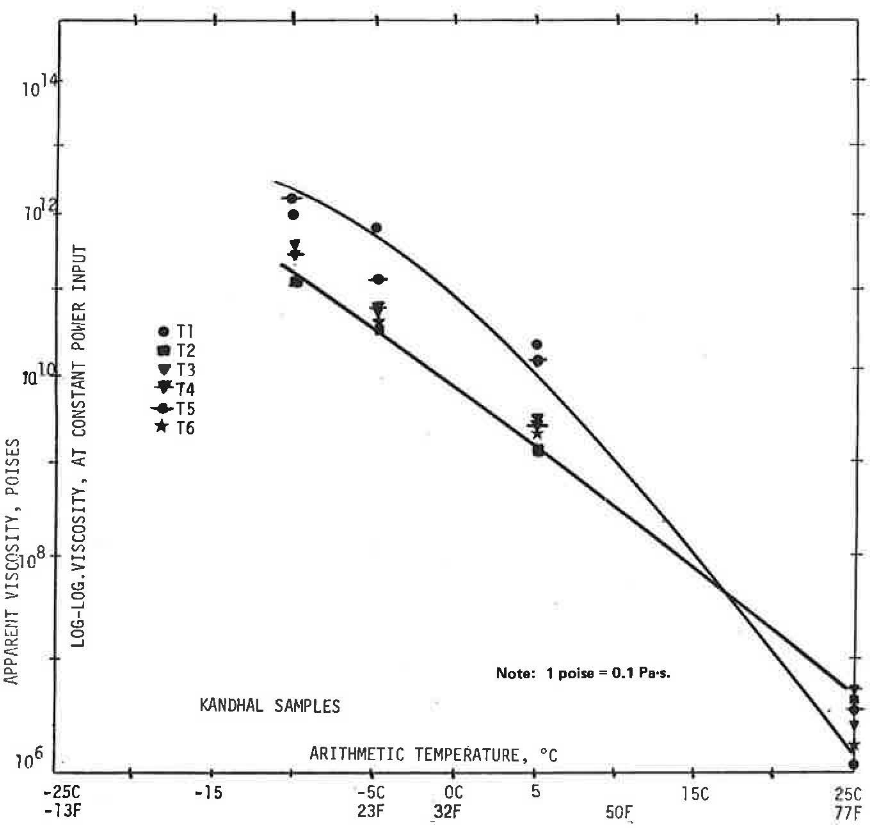

Table 7. Summary of the low-temperature rheology for the six samples.

Temperature (°C)	Property[a]	T-1	T-2	T-3	T-4	T-5	T-6	Mean
25	η (Pa·s x 10^5)	1.09	4.00	2.50	4.71	3.61	1.96	3.98
	G (Pa x 10^7)	3.42	2.48	1.90	5.90	21.1	2.04	6.14
	t_c (s)	0.003	0.016	0.013	0.008	0.002	0.010	0.0086
5	η (Pa·s x 10^8)	20.6	1.67	2.45	2.38	13.3	2.20	7.10
	G (Pa x 10^8)	8.04	5.82	13.4	6.13	10.8	7.28	8.58
	t_c (s)	2.56	0.287	0.183	0.383	1.23	0.302	0.82
-5	η (Pa·s x 10^9)	72.1	4.08	5.80	6.14	29.5	5.80	20.6
	G (Pa x 10^8)	4.14	2.23	2.15	1.37	1.85	4.08	2.64
	t_c (s)	174	18.3	26.9	44.8	159	14.2	72.9
-10	η (Pa·s x 10^{10})	9.40	1.67	4.70	3.16	12.6	2.32	5.64
	G (Pa x 10^8)	3.94	2.34	3.12	3.06	3.47	4.55	3.48
	t_c (s)	238	71.4	151	103	363	51	163

Note: t°C = (t°F -32)/1.8

[a] η = viscosity; G = shear modulus; t_c = time; all G values are in a similar range after passing the transition temperature region.

Table 8. Selected data on mix properties.

Item	T-1	T-2	T-3	T-4	T-5	T-6
Bitumen stiffness modulus[a] at -23°C (MPa)						
At 20 000 s						
Original AC	70.0	14.5	12.0	14.5	25.0	6.0
TFOT residue	58.9	121	74.5	74.2	106	153
Recovered asphalt	145	42.0	36.0	75.0	107	24.0
At 10 000 s	210	62.0	500	110	150	31.2
Viscosity at 20 months (Pa·s)	847	2274	577	800	816	1487
Mix stiffness modulus[a] at -23°C at 20 000 s (MPa)						
Original bitumen	4480	1520	1340	1520	2210	760
Pavement cores	6900	3180	2820	4690	5860	2170
Creep viscosity[b] of laboratory mix at -5°C						
Shear susceptibility index, diametral	0.72	0.79	0.98	1.0	0.78	0.94
Stiffness at 5°C (kPa x 10^6)	10.3	9.11	7.66	9.44	10.8	9.69
Rheology data[c] on original bitumen at -5°C						
Shear susceptibility index	0.72	0.41	0.65	0.47	0.38	0.32
Shear modulus (MPa)	414	223	215	137	185	408
Viscosity (MPa·s)						
η_1	4150	23.2	255	53.0	682	13.3
η_{05}	9730	136	72.2	259	446	102
t_{05} (s)	93	0.61	3.36	1.89	2.41	0.25
η_j at constant power of 100 W/m³ (MPa·s)	72 100	4080	5800	6140	29 500	5800
t_j at constant power of 100 W/m³ (s)	174	18.3	26.9	44.8	159	14.2

Note: t°C = (t°F -32)/1.8.

[a] Van der Poel method.
[b] Run at FDOT on Material Testing System (MTS) machine by Ruth.
[c] Run at the University of Florida on the Schweyer rheometer.

Table 9. Ranges of strength data for mixes.

Temperature (°C)	Tensile Strength (MPa) Range	Tensile Strength Average	Failure Strain (mm/mm x 10^{-4}) Range	Failure Strain Average (%)	Stiffness Modulus (kPa x 10^6) Range	Stiffness Modulus Average
-12	1.6-2.4	2.0	3.6-8.4	0.049	1.0-1.5	1.2
-18	1.9-2.8	2.5	3.4-4.9	0.040	1.3-1.7	1.6
-23	2.2-2.9	2.6	3.2-4.4	0.037	1.2-2.2	1.7
-29	2.4-3.2	2.8	2.9-4.4	0.035	1.8-2.8	2.1

Note: t°C = (t°F -32)/1.8.

stiffness limit times than the others below 5°C (41°F), and (b) we are working with diametral testing of the original mix material at -5°C (23°F) and at 5°C (41°F) to develop a correlation between the mix rheology and bitumen rheology that may help to explain the differences in service.

In connection with the temperature susceptibility, the low sensitivity for the ring and ball softening-point range should be noted. The data indicate a similar range of 49-51°C (120-124°F), which will give about the same PI for a given penetration at a specific AC viscosity grade that allows for testing variance; yet there is a definite variability in consistency. This is apparent on a sensitive viscosity-temperature chart (such as Figure 15) that has a more extended viscosity scale than the American Society for Testing and Materials (ASTM) chart.

Regarding the ductility test at low temperature, it is interesting to note that at the TFOT ductility test's lowest temperature of 4°C (39.2°F) the T-1 and T-5 bitumens that cracked had ductilities higher than or equal to at least one of those in the noncracking pavements under similar test conditions. This was also true at both 15.6°C (60°F) and 25°C (77°F); this should say something about the inadequacy of the ductility test to predict cracking. On the positive side, the data provide evidence that the shear moduli (G_o) appear similar, respectively, for the set of samples. The combination of viscosity and elasticity to give the characteristic time (t_c), as described by Schweyer and coworkers (10,11) at a constant power input of 100 W/m³, separates the samples into two groups at all three low temperatures. The results at 5°C (41°F) and below are summarized in Table 7 for the six samples. The property t_c (also known as the relaxation time) is defined as the ratio of the viscosity divided by the shear modulus at a constant power input to the sample. It represents the time required for the creep viscosity to act and permit the material to flow an amount equal to the initial

Table 10. Comparison of penetration and viscosity as determined by Kandhal and by FDOT data.

	T-1		T-2		T-3		T-4		T-5		T-6	
Property	Kandhal	FDOT	Kandhal	FDOT	Kandhal	FDOT	Kandhal	FDOT	Kandhal	FDOT	Kandhal	FDOT
Original Samples												
Viscosity at 60°C (Pa·s)	271		228		176		171		176		198	
Penetration at 25°C (100 g, 5 s)	42		64		72		65		54		80	
TFOT Residue												
Viscosity at 60°C (Pa·s)	550	494	684	681	398	410	469	454	325	342	572	512
Penetration at 25°C (100 g, 5 s)	26	28	38	42	45	46	38	41	37	38	44	40
Recovered Samples After 20 Months												
Viscosity at 60°C (Pa·s)	847	1152	2274	1394	557	2800	800	1764	817	1037	1487	737
Penetration at 25°C (100 g, 5 s)	19	17	25	28	36	29	31	24	21	19	29	29

Note: The Kandhal data above are from the author's Tables 2 and 4: The viscosity numbers have been rounded off; t°C = (t°F −32)/1.8.

elastic strain that has occurred at a given set of conditions.

Attention is drawn to the t_c values of both the T-1 and the T-5 asphalts, which become greater in absolute value for T-1 and T-5 than for the other asphalts as the temperature is lowered. This is believed to be quite important in connection with low-temperature cracking. If the viscous creep cannot accommodate the elastic deformation before the bitumen reaches its failure strain, the pavement cannot maintain its integrity. It is believed that this particular combination of failure strain and test geometry determines the allowable stress (failure stress) at which the cracking occurs on a level that produces an irreversible effect.

The basic data on construction of the pavements, the temperatures, and the degree of cracking were given in Kandhal's paper (note that T-1 and T-5 had Ontario crack indices of 51/76 and 38/54, respectively). Later data from Kandhal and TFOT data run at FDOT [S_F at 5°C (41°F)] are presented in Table 8. Some comments about the mix properties listed in the table are in order:

1. The bitumen stiffness values found by Kandhal (van der Poel method) are essentially the same magnitude at 1000 MPa at 20 000 s at −23°C (110°F); T-1 and T-5 are higher.
2. Pavement cores for recovered asphalt values at the same conditions as those above show generally slightly higher values but still at the same order of magnitude, although at this level each number differs by 1000 MPa in absolute value.
3. Data on temperature susceptibility in Table 6 show changes that are difficult to interpret. A plot of measured viscosity over the temperature range below 25°C (77°F) appears to be of more interest and value. Such a plot appears in Figure 15.
4. In connection with the tensile strengths and failure strains in Table 5, it should be noted that for all bitumens there is not much variation. The ranges and averages for the four temperatures are summarized in Table 9.
5. The results indicate that, although there is a hardening trend as the temperature is lowered from −12°C (10°F), the average tensile strength varies only from 1.60 to 3.16 MPa.
6. The data shown in Table 8 on stiffness of the original and recovered bitumen as estimated by the van der Poel procedure at −23°C (−10°F) are at about the same level (100 MPa) as those measured at Florida at 5°C (41°F) for the shear modulus.
7. The data for the mixes also show that the Kandhal data at −23°C are at about the same stiffness levels as the Florida data at 5°C.
8. The above items indicate that after the materials enter the glass transition region there does not seem to be too much more hardening on an approximate basis. This is perhaps a logical conclusion, as indicated by the flattening of temperature viscosity plots shown in Figure 15. In addition, the differences may be greater than is apparent since, at a consistency of that level, each integer represents a change of 1000 MPa.

The above information, together with the temperature susceptibility data given in Figure 15, should permit us to determine how the rheology controls the complicated flow behavior of bitumen and, in turn, the mixtures. By using measurements from scientific procedures (instead of by using archaic empirical tests and extensive extrapolations from high-temperature data), we should be able to make better progress in improving highways. This becomes particularly important when mixed crudes from new sources and the use of additives and admixtures in recycling generate new materials whose properties certainly will be different from those of the ones that have been in common use. The work with the Kandhal samples is very interesting and will be explored further and in more detail in the future.

This preliminary comment will be supplemented by further analysis as additional information is developed in this project. As a matter of interest, Table 10 compares Kandhal's data on viscosity at 60°C (140°F) and penetration at 25°C (77°F) with those found by FDOT.

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Author's Closure

I appreciate the additional testing and analyses

performed by Schweyer, Ruth, and Potts on the six asphalt cements used on this project. This is a significant contribution toward understanding the complex behavior of paving asphalts.

It has been noted that the softening points of all asphalt cements fall in a narrow range of 48.9-51.1°C (120-124°F) and thus have a low sensitivity in connection with the temperature susceptibility. However, the softening points should be evaluated in conjunction with the penetrations at 25°C (77°F), which vary from 4.2 to 8.0 mm, in order to determine the relative differences in temperature susceptibility.

Regarding the ductility test at low temperature, the TFOT ductility results (Table 2) did not predict the low-temperature cracking. However, it is interesting to note the ductility values at 4°C (39.2°F) run on the asphalt cements recovered from the project just after construction (Table 3), which indicate very low ductility values for T-1 and T-5 asphalt cements.

It would be interesting to follow up this study to ascertain which of the remaining four asphalt cements develops low-temperature cracking first.

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Seasonal Variation in Skid Resistance of Bituminous Surfaces in Indiana

B. L. ELKIN, K. J. KERCHER, AND S. GULEN

Results are reported of repetitive testing of 15 individual bituminous sections at speeds of 40, 50, and 60 miles/h by means of a skid-resistance measuring system as described by ASTM E274 to identify the surface types that provide and maintain satisfactory skid resistance independent of speed, seasonal changes, and climatic factors such as rainfall and temperature. The bituminous test sections represent surface types commonly used in Indiana, an experimental open-graded friction course, and conventional mixes modified by the substitution of slag for some or all of the conventional aggregate portion. A complete petrographic analysis that concentrated on the carbonates of the coarsest fraction was performed on individual pieces of aggregate extracted from a series of cores taken from the test sections. The report also briefly describes the calibration and standardization of Indiana's skid-resistance measurement system. The cyclic nature of skid resistance relative to season is very apparent for all of the surface types included in the study. With one exception, the skid resistance was highest in the spring, dropped off noticeably during the summer, and began to recover in late fall. Average skid values at 40 miles/h for all the test sections ranged from a high of 61.8 to a low of 23.8. Speed gradients were calculated and compared to provide an indication of seasonal sensitivity. Information obtained from the petrographic analysis and accelerated wear rates determined by means of the British polishing wheel in the laboratory revealed that slag has a greater potential for polishing than the aggregates, which are predominantly limestone. However, skid test results show that the addition of slag improves the frictional characteristics of the pavement. Dolomite appears to be more susceptible to polishing than limestone but is not as susceptible as the slag.

This report was prepared to provide for the early dissemination of information obtained midstream in a study that received Federal Highway Administration (FHWA) approval in December 1976. A sizable quantity of information has been gathered in a program of replicate sampling and testing of 15 selected test sections distributed throughout the state of Indiana. All of the routinely used conventional bituminous surface types are included, as well as some that are experimental because of modification of the mix design, aggregate type, or aggregate gradation. Field testing is to continue through the fall of 1980.

All states are required to perform pavement skid-resistance tests and to develop and provide an inventory of skid resistance for traffic safety purposes (1). Furthermore, most states obtain this information by the use of a skid-test measurement system (SMS) as described by ASTM E274. Once the measurements are obtained, the data may or may not be distributed in some fashion to maintenance, traffic, or traffic safety departments to provide a data base for future analysis or to identify those areas that appear to require immediate attention because they have low skid-resistance values and are high-accident-rate locations.

An inventory system has been established, much testing has been accomplished, and a system of reporting and follow-up has been implemented and is working; however, certain questions need to be answered. More information is needed to allow for the accurate analysis of the data obtained: Are individual test values valid? How many tests are needed to give a true indication of the surface friction under traffic? What factors affect skid-resistance values? What does it mean if there is some disparity between values obtained on what appears to be the same type of surface for the same conditions? How much disparity is normal or acceptable? Why does temperature seem to affect the skid resistance of some pavement types at some times and not at others? In short, the objective is a better understanding of the meaning of pavement skid resistance as measured by a towed trailer system. Therefore, the variability of the skid system itself must be considered. In addition, the effect of the weather, seasons, climate, pavement type and condition, and speed of the vehicle traveling on the roads must be determined. Traffic volumes obviously affect the wear rate of certain pavement types more than others, as do the type and condition of the coarse aggregate fraction incorporated in the pavement surface.

This report describes and documents the preliminary activities and initial analysis of the data collected to date on 15 bituminous sections. This information should be considered at this stage as the foundation for the development of a relationship between pavement skid resistance and seasonal changes, mix design, surface texture, and pavement wear rates that can be used to answer the questions posed earlier.