Evaluation and Field Validation of Proposed Strategic Highway Research Program Binder Specification for Thermal Cracking

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An empirical approach is used to validate the proposed Strategic Highway Research Program (SHRP) binder specification for thermal cracking. Binders extracted from the SHRP A-005 General Pavement Study sections are used to validate the proposed binder specification. The specification parameters at the required test temperatures were generated from the aged binders' master stiffness curves. Although the proposed specification parameters [creep stiffness (S) and slope of the binder stiffness curve (m) at 60-sec loading time] appear to be reasonable, the temperature ranges defined in Version 7G of the specifications are clearly too broad and result in a specification that is too restrictive. The defined limits on S and m are also too restrictive. Alternative limits are suggested for a specification with narrower temperature ranges. The binder specification was subsequently altered as a result of this and other work. The revised specification is also evaluated. The new specification structure and limits appear to be more effective.

The binder specification parameters that have been forwarded as those controlling thermal cracking potential are the creep stiffness (S) and the slope of the binder stiffness curve (m) at 60-sec loading time. The temperature at which the two parameters are obtained, referred to as test temperature, is selected on the basis of the lowest pavement service temperature. The binder specification structure is shown in Figure 1. The validation and evaluation of the binder specification is described in the following sections and is based on Strategic Highway Research Program (SHRP) Binder Specification Draft 7G, as developed under SHRP Contract A-002. The specification is designed for test properties for pressure aging vessel-aged materials.

APPROACH

Binders extracted from the SHRP A-005 General Pavement Study (GPS) sections were used to validate the proposed binder specification. These sections were selected to represent a range of thermal cracking performance (*1*). The sections used are given in Table 1. Both the SHRP code and the code used as a random blind at the Pennsylvania Transportation Institute (PTI) are indicated. The PTI code is used as the section reference throughout this paper. An empirical approach was used to validate the proposed binder specifications. Existing performance models require mixture properties that cannot be determined directly from the binder with existing relationships. First, the specification was applied as forwarded to binder properties obtained from aged field specimens and related to field performance of those sections. Second, the reasons for any discrepancies were examined. The suitability of the specimen parameters was evaluated. Finally, recommendations for changing the specification ranges and limits were formulated to reduce discrepancies between the specification and observed performance.

In this analysis, direct measurements of the specification parameters at the appropriate test temperatures were not available. Therefore, the parameters for the test temperatures were generated from the binders' master stiffness curves and shift functions, using the time-temperature superposition principle.

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PERFORMANCE DATA

A consistent measure of low-temperature cracking performance was needed for each section for the validation. Because of a number of changes and differences in interpretation of the Long-Term Pavement Performance data collection method, the visual surveys could not be directly compared. Reported cracking comparisons between sections and between years on a single section were not consistent. Therefore, the pavement condition data collected by the PASCO video imaging device were used; the data were obtained directly from PCS Law, Inc., where the data were reduced and interpreted.

To correlate field cracking measurements with predicted cracking, a number of assumptions were necessary. All visible cracks were assumed to extend the full depth of the pavement surface; there was no way to differentiate on the basis of the available data. All sections were original pavements; it was assumed that there was no reflective cracking. Cracks were differentiated on the basis of severity in the data, but this information was not directly used in the analysis. The mechanism by which cracks progress from low to high severity also involves mechanisms other than low-temperature contraction. Deteriorative factors include moisture and drainage characteristics, asphalt and mixture properties, and traffic.

Cracking was considered at only one point in time; no cracking versus time data were available because of the changes in data collected and delays in reduction and availability of the PASCO data. The PASCO data were assumed to have picked up all low-temperature cracking and not to have imaged any cracks that were not present. These assumptions regarding the reliability of the

Performance Grade	PG 1-		PG 2-					PG 3-					PG 4-	
	4	5	1	2	3	4	5	1	2	3	4	5	1	2
Average 7-day Maximum Pavement Temperature, °C[a]	<45		<55					<65					<75	
Minimum Pavement Service Temperature, °C[a]	>-30	>-40	>0	>-10	>-20	>-30	>-40	>0	>-10	>-20	>-30	>-40	>0	>-10
Original Binder														
Flash Point Temp, ASTM D 92: Minimum, °C	230													
Viscosity, ASTM D 4402 (Brookfield):[b] Max, 2 Pa • s (2000 cSt) Test Temp, °C	165													
Dynamic Shear, SHRP B-003: G*/sin δ, Min, 1.0 kPa (0.145 psi) Test Temp @ 10 rad/s, °C	45		55					65					75	
Rolling Thin Film Oven Test (AASHTO T 240; ASTM D 2872) Residue[c]														
Max Loss, Max, percent	1.00													
Dynamic Shear, SHRP B-003: G*/sin δ, Min, 2.0 kPa (0.290 psi) Test Temp @ 10 rad/s, °C	45		55					65					75	
Pressure Aging Vessel Residue (SHRP B-005)*														
PAV Aging Temperature, °C	90		100					100					110	
Dynamic Shear, SHRP B-003: G* sin δ, Max, 3000 kPa (435 psi) Test Temp @ 10 rad/s, °C	10	5	30	25	20	15	10	35	30	25	20	15	40	35
Creep Stiffness, SHRP B-002:[d] S, Max, 2 x 10^5 kpa, (29000 psi) m-value, Min, 0.35 Test Temp @ 60s, °C	-20	-30	10	0	-10	-20	-30	10	0	-10	-20	-30	10	0
Direct Tension, SHRP B-006: Failure Strain, Min, 1.0% Test Temp @ 1.0 mm/min, °C	-20	-30	10	0	-10	-20	-30	10	0	-10	-20	-30	10	0

Notes: [a] Pavement temperatures determined from air temperatures using algorithm contained in SUPERPAVE program.
[b] AASHTO T 2020, (ASTM D 2171) may be used in lieu of (ASTM D 4402), however, ASTM D 4402 is considered reference method. A second viscosity measurement (145 °C) may be required for reporting purposes to develop viscosity-temperature relationship for estimating mixing and compaction temperatures.
[c] TFOT AASHTO T 179 (ASTM D 1754) may be used in lieu of AASHTO T 240, (ASTM D 2872) is considered reference method.
[d] S is stiffness after 60 seconds loading time and m is the slope of the log stiffness versus log time curve at 60 seconds loading time.

Comments:
1. Tenderness is related to the values of G*/sin δ before and after RTFOT.
2. Rutting is related to the value of G*/sin δ after RTFOT.
3. Fatigue is related to the value of G* sin δ and direct tension strain-to-failure after PAV.
4. Low-temperature thermal cracking is related to S, m, and direct tension strain-to-failure after PAV.
5. Rheological type is controlled by m.

FIGURE 1 SHRP Binder Specification Draft 7G.

PASCO data were discussed with PCS Law personnel who reduced the data and are consistent with the digitizing criteria.

The PASCO data were received in a digitized format, including a summary of pavement distresses for each section. Observation of the crack maps indicated that the amount of cracking was misrepresented by looking only at the number of feet of cracking. The cracking pattern and likelihood that the cracks were caused by low-temperature stresses are also important. All of the transverse and longitudinal cracking is not due to low-temperature cracking; for example, low-severity longitudinal cracking in the wheelpath is probably early-stage fatigue cracking. Other small "stray" cracks were also not considered to be due to low-temperature distress.

A rational attempt was made to estimate the amount of cracking caused by thermal stresses using the available data. The following observations and assumptions were made:

- It appeared more rational to estimate the amount of thermal cracking by multiplying the number of complete or near complete transverse cracks by the width of the lane [generally 3.65 m (12 ft)] than by using the linear sum of all cracking recorded in the PASCO data.
- It was also observed that the pavement sections could be grouped into one of four categories of thermal cracking (zero, low, medium, or high), within which it was difficult to distinguish between the thermal cracking performance of the sections. On the basis of the range in amount of cracking observed within each category, the following system was established to categorize the pavement section with respect to thermal cracking performance: zero cracking, 0 to 7.6 m (25 ft) of cracking per 152-m (500-ft) section [less than one crack per 76 m (250 ft)]; low cracking, 7.6 to 22.9 m (25 to 75 ft) of cracking per 152-m (500-ft) section [from one crack per 76 m (250 ft) to one crack per 25.9 m (85

ft)]; medium cracking, 22.9 to 45.7 m (75 to 150 ft) of cracking per 152-m (500-ft) section [from one crack per 25.9 m (85 ft) to one crack per 12.2 m (40 ft)]; and high cracking, greater than 45.7 m (150 ft) of cracking per 152-m (500-ft) section [more than one crack per 12.2 m (40 ft)].

SERVICE TEMPERATURES

Weather data were obtained from the National Climatic Data Center (NCDC) and from the SHRP A-001 contractor. The greatest emphasis was placed on obtaining reliable daily temperatures for each section. The temperatures were obtained for the closest available weather station with relatively complete data for the years since the pavement was placed. For cases in which several weather stations were approximately equidistant and the temperature information for those stations was different, additional evaluation was completed to select the most appropriate weather station. Topographic maps were consulted and the pavement section and weather stations located. Elevations for the sites were compared, and any intervening topographic features were noted. The most appropriate station was then identified, as given in Table 1.

The minimum pavement surface temperatures of these sections are between −11°C and −31°C, which represents a broad range of service temperatures. The minimum pavement service temperatures were determined using actual air temperatures from NCDC weather stations near the pavement sections for analysis with the FHWA Integrated Environmental Effects Model. The FHWA model was developed for FHWA's Office of Engineering and Highway Operations Research and Development by the Texas Transportation Institute, Texas A&M University (*2*). The use of this model for determining minimum pavement service temperatures for low-temperature cracking predictions has been demonstrated (*3*).

TABLE 1 GPS Sections and Weather Stations Used

State	PTI Code	SHRP Code	Weather Station	Station ID
AZ	2	41022	Hackberry	9158
CO	18	81047	Rangely	6832
ID	12	161001	Coeur D Alene	1956
ID	11	161010	Idaho Falls	4457
IN	32	181028	Huntingburg	7724
IN	36	181037	Bonneville	2731
KS	7	201005	Ottawa	6128
KY	21	211034	Glasgow	3246
ME	33	231026	Farmington	2765
MD	23	241634	Berlin	335
MN	38	271028	Frazee	2142
MN	37	271087	Farmington	5435
MO	28	291010	Waynesville	8777
NB	13	311030	Edison	640
NV	6	322027	Oasis	2573
NJ	27	341011	Trenton	3951
OK	1	404086	Chickasha	1750
OK	22	404088	Ponca City	7201
PA	31	421597	Lawrenceville	5408
SC	26	451008	Salem	8887
TX	3	481183	Southland	7206
UT	16	491008	Marysvale	3514
WY	17	561007	Cody	1840

SELECTION AND EXTRACTION OF BINDER SPECIMEN

The asphalt cement from each of the 23 GPS sites used in this study was extracted from the tested cores and analyzed to determine its linear viscoelastic properties. The extraction procedure used in this project was developed by the SHRP binder study group. To properly characterize each of the field sections used in this study, it was necessary to slice each of the field cores according to their individual cross-sectional characteristics. Figure 2 shows the basic approach used to obtain representative materials for testing from the field cores. In many cases, more than one mixture was present in the cross section. The approach taken was to test the binder from the mixture that predominantly controls the thermal cracking performance of a particular cross section. Obviously, detailed testing at multiple levels of all materials present in the cross section would have been desirable. However, constraints in terms of both time and resources did not allow for such a comprehensive testing program. Three cases were identified and slices were obtained for extractions as indicated in Figure 2.

DEVELOPMENT OF BINDER MASTER CURVES

The extracted material was characterized according to its linear viscoelastic parameters as recommended by SHRP A-002. Two test devices were used to obtain the data required to characterize the different asphalt cements obtained from the tested cores. The first was a bending beam rheometer test (*4*). This test provided the flexural creep stiffness, S, and the log-log slope of the creep curve, m, at −15°C and 120 sec. The second device was a dynamic shear rheometer. From this test, the complex modulus, G^*, at 15°C, 25°C, 45°C, and 60°C was determined for frequencies, f, of 15, 8, 5, 3, 1.5, 0.8, 0.5, 0.3, and 0.15 Hz.

The test results from both testing devices and each of the five temperatures were combined into a master stiffness curve that characterizes the viscoelastic properties of the material using the procedure recommended by Christensen and Anderson (*5,6*). The first step consisted of transforming the magnitude of the complex modulus, G^*, into creep stiffness, S. An exact transformation between these two material responses would require complicated numerical transformation. For practical purposes, it was decided to use a simple and very common approach of assuming that both variables are related at any given frequency by Poisson's ratio, μ, as follows:

$$E = 2(1 + \mu)G^* \quad (1)$$

If it is assumed that the asphalt cement is incompressible and thus has a Poisson's ratio of 0.50, the following conversion is obtained:

$$E = 3G^* \quad (2)$$

This conversion, although not exact, is considered a very reasonable approximation by most researchers. It is reported to be sufficiently accurate for engineering calculations with a maximum error of less than 15 percent (*5*).

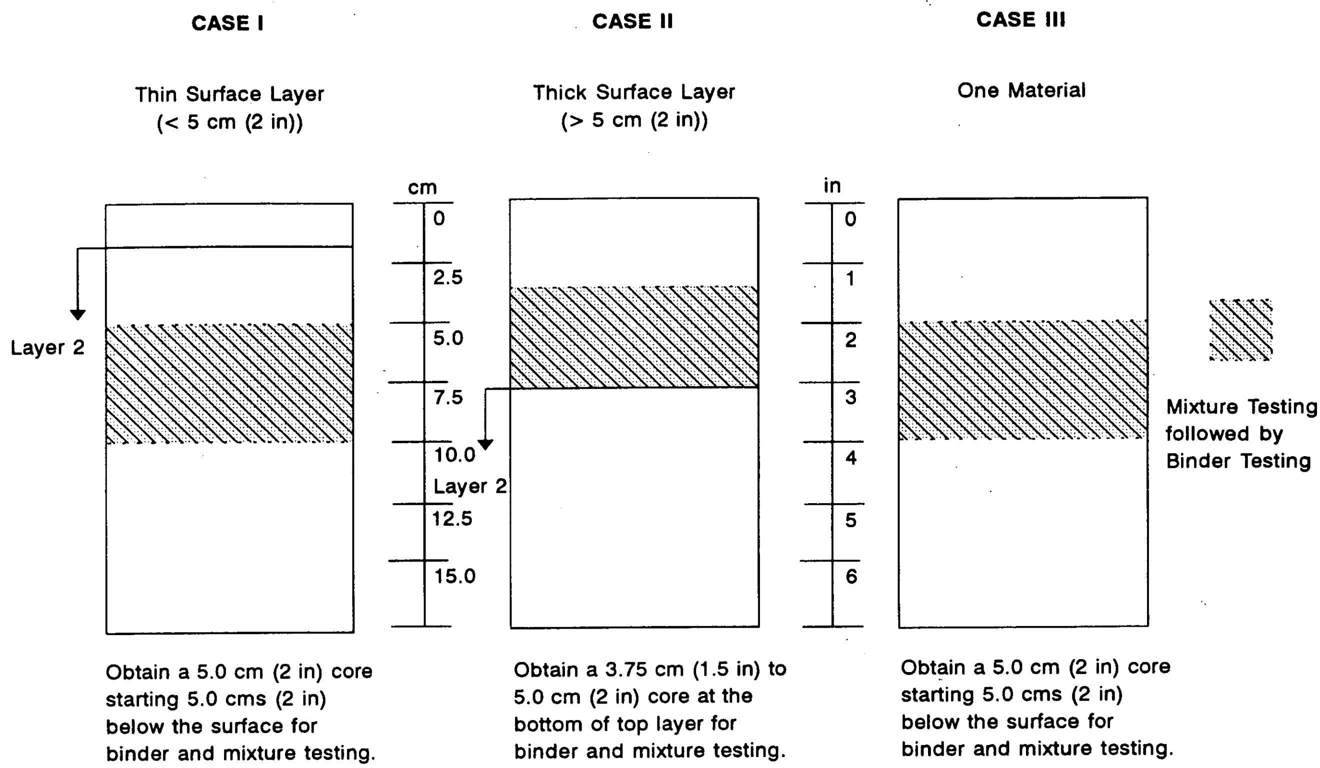

FIGURE 2 Selection of binder specimen.

The test frequencies were converted to loading time using the following formula:

$$t = 1/(2\pi f) \tag{3}$$

Once all the conversions were made, a nonlinear regression routine was performed to fit the data into the following equation proposed by Christensen (*6*):

$$S_{(t,T)} = S_g\left[1 + \left(\frac{t_r}{t_0}\right)^{\frac{\log 2}{R}}\right]^{-\frac{R}{\log 2}} \tag{4}$$

where

$S_{(t,T)}$ = stiffness at loading time t and reference temperature T;
S_g = glassy stiffness, assumed to be 3 GPa for all asphalt cements;
t_r = reduced time (sec);
t_0 = crossover time (sec) (this value is very close to the point at which the viscous asymptote crosses the glassy stiffness and is considered a hardness parameter); and
R = rheological index, defined as the difference between the glassy stiffness and the stiffness at the crossover time.

The reduced time, t_r, and the loading time, t, are related by the time temperature superposition shift factors, a_T, as follows:

$$t_r = t^* (a_T/a_{T\text{ref}}) \tag{5}$$

where $a_T/a_{T\text{ref}}$ is the shift of the time t at temperature T to the reference temperature T_{ref}. The shift factors were represented using two equations. The first of these equations, the Williams-Landel-Ferry (WLF) equation, is used above the defining temperature T_d:

$$\log \frac{a_T}{a_{T\text{ref}}} = \frac{-C_1(T - T_{\text{ref}})}{C_2 + T - T_{\text{ref}}} \tag{6}$$

where C_1 and C_2 are empirically determined constants that were fixed as 19 and 92, respectively. In the regression, to ease numerical computations, the reference temperature T_{ref} was assumed to be the defining temperature T_d.

The second equation is an Arrhenius function, which is used to describe the shift factors as a function of temperature below the defining temperature T_d:

$$\log \frac{a_T}{a_{T\text{ref}}} = \frac{H_a}{2.303R}\left(\frac{1}{T} - \frac{1}{T_{\text{ref}}}\right) \tag{7}$$

where H_a is the activation energy for flow below T_d, fixed as 250 KJ/mol, and R is the ideal gas constant, 8.34 J/mol-°K.

A detailed explanation of these models and the meaning of each of the parameters used is given by Christensen (*6*).

The regressed coefficients for the binders extracted from each of the 23 GPS sections are presented in Table 2. The resulting master stiffness curves were plotted; an example is shown as Figure 3. The binder specification parameters were obtained from these master stiffness curves.

TABLE 2 Regressed Coefficients for Extracted Binders

PTI Code	Model Fitting Results at T_d			
	S_g (Pa)	Log t_0 (sec)	R	T_d (°C)
1	3.00 x 10^9	2.52	1.36	-14.55
2	3.00 x 10^9	2.03	0.89	-13.90
6	3.00 x 10^9	2.12	1.42	-15.00
7	3.00 x 10^9	5.27	2.98	-12.90
11	3.00 x 10^9	2.93	1.54	-15.00
12	3.00 x 10^9	3.06	1.82	-12.85
13	3.00 x 10^9	2.90	2.49	-13.23
16	3.00 x 10^9	2.19	1.33	-13.12
17	3.00 x 10^9	2.20	1.41	-15.02
18	3.00 x 10^9	2.36	1.23	-14.97
21	3.00 x 10^9	2.26	1.77	-12.15
22	3.00 x 10^9	3.23	2.26	-12.68
23	3.00 x 10^9	3.31	1.90	-14.04
26	3.00 x 10^9	4.56	1.94	-14.90
27	3.00 x 10^9	2.38	1.62	-14.38
28	3.00 x 10^9	2.16	1.05	-13.47
31	3.00 x 10^9	3.19	1.71	-12.85
32	3.00 x 10^9	2.52	1.74	-13.46
33	3.00 x 10^9	2.07	1.74	-14.33
36	3.00 x 10^9	2.42	1.39	-14.71
37	3.00 x 10^9	2.57	1.67	-14.38
38	3.00 x 10^9	3.70	1.73	-14.61

VALIDATION OF BINDER SPECIFICATION VERSION 7G

An empirical approach was used to validate the proposed binder specifications. The parameters S and m at 60-sec loading time and the appropriate test temperatures were obtained for the 22 GPS sections used in this project. Those properties were compared with the specification limits proposed for thermal cracking. The failed and passed sections, as determined by the specifications, were then compared with the actual cracking observed in the field. Table 3 gives the comparisons between observed cracking and the potential for cracking as predicted by the binder specifications.

Table 3 indicates that only four of the 22 binders passed the specification requirements. Further examination indicates that the actual cracking observations in the field do not agree with the specification findings. Some binders used in sections with medium observed cracking passed the specification requirements. Conversely, sections with zero and low observed cracking had binder properties that failed the specification. In fact, 7 of the 18 binders that failed the specifications were in sections where either zero or low cracking was observed in the field. All sections with observed high cracking failed the specification. These observations warranted further evaluation and investigation of the proposed specification.

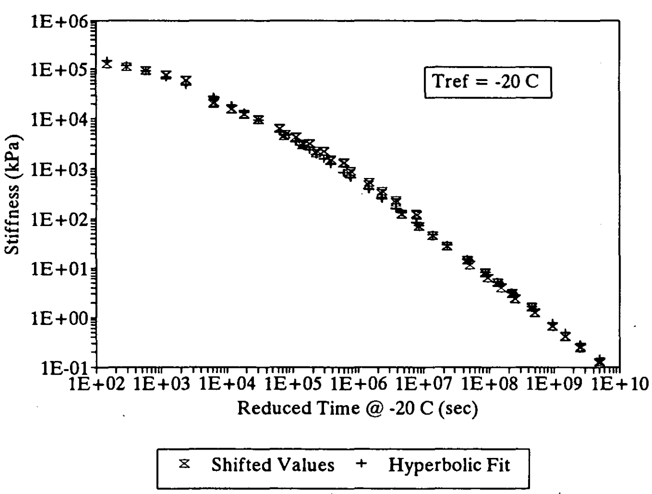

FIGURE 3 Example binder master stiffness curve.

EVALUATION OF THE SPECIFICATION FORMAT

Additional work was performed to investigate the specification limits and temperature ranges proposed in the specifications. The sections were categorized into four groups on the basis of S and m values as determined by the specifications to check for a relationship between failure in either of the specification parameters and cracking potential. The four categories were established on the basis of magnitudes of S and m at the test temperature relative to the specification limits for S and m. The specification limits have been established as a maximum of 2×10^5 kPA (29,000 psi) for S and a minimum of 0.35 for m. The following four categories are indicated in Table 4:

- Sections with binders that have S less than 2×10^5 kPA and m more than 0.35,
- Sections with binders that have S more than 2×10^5 kPA and m more than 0.35 (no sections),
- Sections with binders that have S less than 2×10^5 kPA and m less than 0.35, and
- Sections with binders that have S more than 2×10^5 kPA and m less than 0.35.

Table 4 indicates that almost all the sections rejected by the specification did not meet the specification limits for both S and m. On the other hand, none of the sections was rejected on the basis of the S requirement alone. Table 4 also indicates that two sections did not pass the specification requirement for m only.

All the binders used in the remaining GPS sections not passing the specifications failed to meet both the S and the m limits; neither parameter is solely responsible for rejecting these binders. The data reported in Table 4 indicate that there is no clear correlation between adherence to either of the specification limits and the observed cracking in the field.

A key observation was made with respect to the test temperatures at which these parameters were obtained; it can be clearly seen that binders with test temperatures of −10°C passed the specifications. Section 26 is the only exception to this statement; however, this section is very close to passing the limits. On the other hand, all sections with parameters obtained at test temperatures of −20°C or −30°C failed to meet the specification limits. The outcome of the specification, in its current format, is primarily driven by the test temperature. It appears that the current specification structure is not as sensitive to differences between different binder properties as it is to differences between test temperatures.

TABLE 3 Evaluation of Binders Extracted from GPS Sections with SHRP Binder Specifications, Draft 7G

Specification Result	PTI Code	Minimum Pavement Temperature (°C)	Test (Evaluation) Temperature (°C)	Stiffness @ 60 sec (kPa)	m @ 60 sec	Observed Thermal Cracking (Field) Level of Cracking	(m/152 m)	(ft/500 ft)
PASS	1	-19	-10	109351	0.48	Medium	29	96
	2	-19	-10	199738	0.55	Zero	0	0
	22	-19	-10	43873	0.41	Medium	29	96
	23	-14	-10	82986	0.41	Zero	0	0
FAIL	6	-27	-20	41556	0.31	High	>61	>200
	7	-24	-20	191637	0.23	High	>61	>200
	11	-34	-30	1488327	0.12	High	>61	>200
	12	-21	-20	470746	0.24	Zero	>61	>200
	13	-28	-20	102530	0.32	Low	11	36
	16	-27	-20	684663	0.26	High	>61	>200
	17	-32	-30	1401117	0.13	High	>61	>200
	18	-27	-20	737556	0.26	High	>61	>200
	21	-20	-20	334899	0.29	Zero	0	0
	26	-14	-10	200479	0.32	Medium	29	96
	27	-20	-20	364195	0.30	Low	11	36
	28	-25	-20	1106451	0.22	Medium	37	120
	31	-24	-20	624240	0.22	Low	7	24
	32	-23	-20	358349	0.29	Low	4	12
	33	-28	-20	222508	0.34	Low	4	12
	36	-21	-20	575941	0.27	High	>61	>200
	37	-29	-20	377666	0.29	High	40	132
	38	-34	-30	1492515	0.10	High	>61	>200

TABLE 4 Consideration of Limiting Binder Specification Criteria

Category	PTI Code	Minimum Pavement Temperature (°C)	Test (Evaluation) Temperature (°C)	Stiffness @ 60 sec (kPA)	m @ 60 sec	Observed Thermal Cracking (Field)	Age (years)	Thickness (cm)
Low S	1	-19	-10	109354	0.48	Medium	19	41.9
High m	2	-19	-10	199738	0.55	Zero	13	41.9
	22	-19	-10	43873	0.41	Medium	20	31.8
	23	-14	-10	82986	0.41	Zero	14	8.9
Low S	7	-24	-20	191637	0.23	High	17	33.0
Low m	13	-28	-20	102530	0.32	Low	8	19.1
High S	6	-27	-20	415566	0.31	High	13	23.5
Low m	11	-34	-30	1488327	0.12	High	19	27.3
	12	-21	-20	470746	0.24	Zero	15	8.9
	16	-27	-20	684663	0.26	High	16	23.5
	17	-32	-30	1401117	0.13	High	9	8.9
	18	-27	-20	737556	0.26	High	7	10.2
	21	-20	-20	334899	0.29	Zero	18	36.2
	26	-14	-10	200479	0.32	Medium	20	12.1
	27	-20	-20	364609	0.30	Low	20	23.5
	28	-25	-20	1106451	0.22	Medium	10	33.0
	31	-24	-20	624240	0.22	Low	10	15.9
	32	-23	-20	358349	0.29	Low	15	38.1
	33	-28	-20	222508	0.34	Low	17	15.9
	36	-21	-20	575941	0.27	High	15	35.6
	37	-29	-20	377666	0.29	High	11	40.6
	38	-34	-30	1492515	0.10	High	18	24.1

TABLE 5 Evaluation of Binders Extracted from GPS Sections with SHRP Binder Specification Draft 7G, with Temperature Intervals Reduced to 1°C

Category	PTI Code	Minimum Pavement Temperature (°C)	Test (Evaluation) Temperature (°C)	Stiffness @ 60 sec (kPA)	m @ 60 sec	Observed Thermal Cracking (Field)	Age (years)	Thickness (cm)
Low S	1	-19	-9	95246	0.50	Medium	19	41.9
High m	2	-19	-9	141695	0.59	Zero	13	41.9
	12	-21	-11	110725	0.40	Zero	15	8.9
	21	-20	-10	53155	0.47	Zero	18	36.2
	22	-19	-9	38134	0.42	Medium	20	31.8
	23	-14	-4	23090	0.51	Zero	14	8.9
	26	-14	-4	81505	0.40	Medium	20	12.1
	27	-20	-10	51475	0.50	Low	20	23.5
	32	-23	-13	107162	0.41	Low	15	38.1
	33	-28	-18	156212	0.37	Low	17	15.9
	36	-21	-11	97743	0.49	High	15	35.6
High m								
High S	6	-27	-17	262416	0.37	High	13	23.5
Low S	7	-24	-14	96446	0.28	High	17	33.0
Low m	13	-28	-18	77739	0.34	Low	8	19.1
High S	11	-34	-24	885773	0.19	High	19	27.3
Low m	16	-27	-17	425262	0.33	High	16	23.5
	17	-32	-22	636645	0.26	High	9	8.9
	18	-27	-17	490611	0.32	High	7	10.2
	28	-25	-15	551378	0.34	Medium	10	33.0
	31	-24	-14	287370	0.31	Low	10	15.9
	37	-29	-19	317016	0.31	High	11	40.6
	38	-34	-24	935777	0.17	High	18	24.1

TABLE 6 Sensitivity of Binder Specification Criteria to Evaluation Temperature

	Test Temperature							
	@ -10 °C (From MSC & a_T)		@ -15 °C (Measured)		@ -20 °C (From MSC & a_T)			
PTI Code	Stiffness @ 60 sec (kPa)	m @ 60 sec	Stiffness @ 60 sec (kPa)	m @ 60 sec	Stiffness @ 60 sec (kPa)	m @ 60 sec	Minimum Pavement Temperature (°C)	Observed Thermal Cracking
1	109351	0.48	304579	0.36	667978	0.26	-16	Medium
2	199741	0.55	643705	0.47	1319084	0.21	-11	Zero
6	51718	0.54	146379	0.41	415568	0.31	-20	High
7	51387	0.32	129073	0.32	191640	0.23	-21	High
11	105117	0.45	227179	0.30	576691	0.25	-30	High
12	97409	0.41	N/A	N/A	470746	0.24	-18	Zero
13	16513	0.45	61443	0.41	102532	0.32	-22	Low
16	109275	0.49	160865	0.38	684663	0.26	-21	High
17	58502	0.53	157469	0.45	450359	0.30	-25	High
18	109730	0.52	250004	0.34	737553	0.26	-22	High
21	53827	0.47	123327	0.44	334899	0.29	-18	Zero
22	43871	0.41	80252	0.40	225852	0.27	-17	Medium
23	82985	0.41	228834	0.40	408521	0.25	-11	Zero
26	200479	0.32	362413	0.23	685180	0.19	-12	Medium
27	52800	0.50	168618	0.40	364609	0.30	-15	Low
28	184028	0.51	558806	0.32	1106450	0.22	-21	Medium
31	141880	0.39	233881	0.36	624238	0.22	-20	Low
32	58019	0.47	98912	0.37	358348	0.29	-21	Low
33	27558	0.53	103790	0.49	222508	0.34	-23	Low
36	87170	0.50	238965	0.33	575940	0.27	-18	High
37	59026	0.48	173338	0.37	377667	0.29	-23	High
38	159752	0.37	340074	0.26	660207	0.21	-31	High

The temperature ranges used in the specification were examined to determine whether the sensitivity to differences in binder properties can be improved. The S and m parameters were obtained at the lowest pavement temperature plus 10°C. This represents temperature ranges of 1°C. This approach resulted in acceptance of most of the binders used in sections with observed low cracking as indicated in Table 5. Seven of the 11 field test sections composed of binders with $S < 2 \times 10^5$ kPA (29,000 psi) and $m > 0.35$ were zero or low cracking sections in the field. Three were medium cracking and one was high cracking. Nine of the 11 field test sections exceeding the specification limits [$S > 2 \times 10^5$ kPA (29,000 psi) or $m < 0.35$] were either medium or high cracking sections in the field. Two were low cracking sections. Considering mixture effects, which may result in variance in performance for a single binder, this indicates that S and m are reasonable parameters.

Table 6 provides additional evidence on the effects of test temperature. Binders from each section were evaluated at −10°C, −15°C, and −20°C. Almost all binders passed at −10°C, and all binders failed at −20°C. Large changes in S and m were observed with a 5°C change in test temperature. Furthermore, Binder Specification 7G indicates that no asphalt can be used in service temperatures below −20°C, whereas available performance data indicate that many of the binders were adequate.

CONCLUSIONS

The empirical correlations performed in this study indicate that the binder specification limits and temperature ranges are inadequate. The primary conclusions regarding Binder Specification 7G are as follows:

- S and m appear to be reasonable binder parameters for controlling thermal cracking.
- It appears that the specification is too restrictive.
- The defined limits on S and m [$S < 2 \times 10^5$ kPA (29,000 psi) and $m > 0.35$] are too restrictive.
- It appears that the temperature ranges (−10°C) used to define different asphalt grades are too broad.
- Thermal cracking performance of a particular binder is mixture dependent and cannot be controlled from binder properties alone.

RECOMMENDATIONS

The philosophy of the binder specification needs to be clearly established. Low-temperature properties can be mixture dependent. Mixture properties such as aggregate type, VMA, compaction, and coefficient of thermal contraction have strong effects on thermal performance. Two mixtures with the same binder may have very different low-temperature properties and resulting thermal cracking performance. The best one can hope for from a binder specification is to reject only those binders that have no chance of performing well regardless of what mixture they are used with. Conversely, the specification should accept binders that have a reasonable chance of performing well with a suitable mixture design. It is, however, very difficult to evaluate the binder specifications given the confounding of the mixture effects.

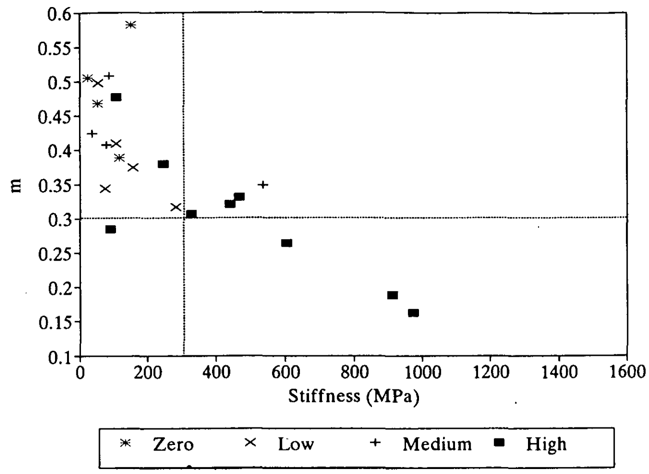

FIGURE 4 Proposed binder specification limits compared with thermal cracking performance of GPS sections for temperature ranges of 1°C.

Although S and m appear to be reasonable, the temperature ranges (−10°C) defined in Version 7G of the binder specifications are clearly too broad and result in a specification that appears to be far too restrictive. These limits should be reduced to as narrow a range as practical, but should probably be no greater than 5°C.

The defined limits on S and m [$S < 2 \times 10^5$ kPA (29,000 psi) and $m > 0.35$] are too restrictive. On the basis of the data obtained in this investigation and as shown in Figure 4, the following limits are recommended: $S < 3.2 \times 10^5$ kPA (45,000 psi) and $m > 0.30$, for temperature ranges of 1°C. Broader temperature ranges may require even less restrictive specification limits. These proposed new limits are consistent with the binder philosophy recommended above.

Further investigations beyond the scope of this project are recommended to verify that S and m are appropriate specification parameters. Such investigations should use mechanistic prediction models to isolate their effects from the effects of other factors such as mix design and pavement geometry. Then the specification structure and corresponding limits could be examined by using performance prediction models.

REVISED SPECIFICATION

As a result of examination of the proposed specification, including the work described in this paper, the binder specification was revised. The temperature ranges for grades were reduced to 6°C; the limits were changed to a maximum stiffness of 300 MPa and a minimum m of 0.30. The revised specification is shown in Figure 5.

Stiffnesses and m values were generated for each of the binders at the temperatures required by the revised specification. Ten binders passed the specification requirements; 12 did not. As shown in Figure 6, two binders from high-cracking sections passed the specification; one binder from a zero-cracking section and three from low-cracking sections failed. The revised specification represents a significant improvement. Although narrower temperature ranges might produce better results, the difficulty of accurately

PERFORMANCE GRADE	PG 52-							PG 58-					PG 64-					PG 70-			
	10	**16**	**22**	**28**	**34**	**40**	**46**	**16**	**22**	**28**	**34**	**40**	**16**	**22**	**28**	**34**	**40**	**10**	**16**	**22**	**28**
Average 7-day Maximum Pavement Design Temperature, °C[a]	<52							<58					<64					<70			
Minimum Pavement Design Temperature, °C[a]	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28
ORIGINAL BINDER																					
Flash Point Temp, T48: Minimum °C	230																				
Viscosity, ASTM D 4402:[b] Maximum, 3 Pa • s, Test Temp, °C	135																				
Dynamic Shear, TP5:[c] $G^*/\sin\delta$, Minimum, 1.0 kPa Test Temp @ 10 rad/s, °C	52							58					64					70			
Physical Hardening Index[d], h	Report																				
ROLLING THIN FILM OVEN RESIDUE (T240)																					
Max Loss, Maximum, percent	1.00																				
Dynamic Shear, TP5: $G^*/\sin\delta$, Minimum, 2.2 kPa Test Temp @ 10 rad/s, °C	52							58					64					70			
PRESSURE AGING VESSEL RESIDUE (PP1)																					
PAV Aging Temperature, °C	90							100					100					100(110)[e]			
Dynamic Shear, TP5: $G^*\sin\delta$, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	25	22	19	16	13	10	7	25	22	19	16	13	28	25	22	19	16	34	31	28	25
Creep Stiffness, TP1:[f] S, Maximum, 300 MPa, m-value, Minimum, 0.30 Test Temp @ 60s, °C	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-6	-12	-18	-24	-30	0	-6	-12	-18
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	-6	-12	-18	-24	-30	0	-6	-12	-18

[a] Pavement temperatures are estimated from air temperatures using an algorithm contained in the SUPERPAVE software program or may be provided by the specifying agency.
[b] This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.
[c] For quality control of unmodified asphalt cement production, measurement of the viscosity of the orignal asphalt cement may be substituted for dynamic shear measurements of $G^*/\sin\delta$ at test temperatures where the asphalt is a Newtonian fluid (generally above 55°C). Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometry.

[d] The physical hardening index h accounts for physical hardening and is calculated by $h=\left(\frac{S_{24}}{S_1}\right)^{\frac{m_1}{m_{24}}}$ where 1 and 24 indicate 1 and 24 hours of conditioning of the tank asphalt. Conditioning and testing is conducted at the designated test temperature. Values should be calculated and reported. S is the creep stiffness after 60 seconds loading time and m is the slope of the log creep stiffness versus log time curve after 60 seconds loading time.
[e] The PAV aging temperature is 100°C, except in desert climates, where it is 110°C.
[f] If the creep stiffness is below 300 MPa, the direct tension test is not required. If the creep stiffness is between 300 and 600 MPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The m-value requirement must be satisfied in both cases.

FIGURE 5 Revised SHRP binder specification, May 1993.

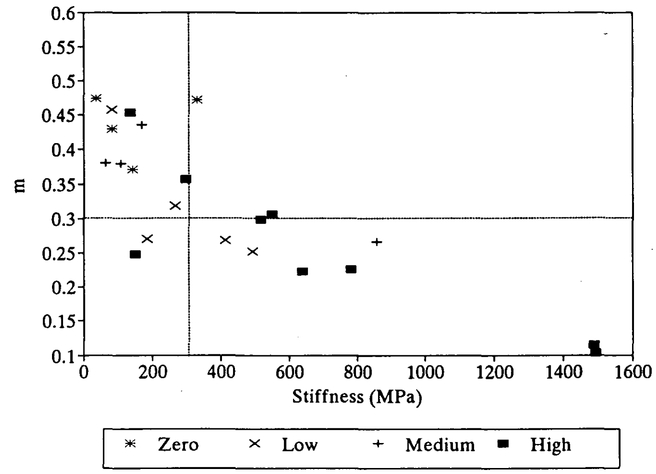

FIGURE 6 Evaluation of binder extracted from GPS sections with revised SHRP binder specification, May 1993.

predicting expected minimum service temperatures may make narrower ranges impractical.

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REFERENCES

1. Lytton, R. L., R. Roque, J. Uzan, D. R. Hiltunen, E. Fernando, and S. M. Stoffels. *Development and Validation of Performance Prediction Models and Specifications for Asphalt Binders and Paving Mixes.* SHRP-A-357. Strategic Highway Research Program, National Research Council, 1993.
2. Lytton, R. L., D. E. Pufahl, C. H. Michalak, H. S. Liang, and B. J. Dempsey. *An Integrated Model of the Climatic Effects on Pavements.* Report 033. Texas Transportation Institute, Texas A&M University, 1993.
3. Stoffels, S. M., W. R. Lauritzen, and R. Roque. Estimation of Asphalt Concrete Pavement Temperatures for the Prediction of Low-Temperature Cracking. In *Transportation Research Record 1417*, TRB, National Research Council, Washington, D.C., 1993, pp. 160–169.
4. Bahia, H. U., D. A. Anderson, and D. W. Christensen. The Bending Beam Rheometer; A Simple Device for Measuring Low-Temperature Rheology of Asphalt Binders. *Journal of the Association of Asphalt Paving Technologists*, Vol. 61, 1992, pp. 117–153.
5. Christensen, D. W., and D. A. Anderson. Interpretation of Dynamic Mechanical Test Data for Paving Grade Asphalt Cements. *Journal of the Association of Asphalt Paving Technologists*, Vol. 61, 1992, pp. 67–116.
6. Christensen, D. W. *Mechanical Modeling of the Linear Viscoelastic Behavior of Asphalt Cements.* Ph.D. dissertation. The Pennsylvania State University, University Park, 1992.

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