PREDICTING LOW-TEMPERATURE CRACKING FREQUENCY OF ASPHALT CONCRETE PAVEMENTS

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Low-temperature shrinkage cracking of asphalt pavements is a serious and costly problem throughout much of Canada and the northern United States. The large amount of effort devoted to the problem has provided several design approaches. This paper describes the latest and most comprehensive of such a design approach. A mathematical model based on field observations has been developed and is capable of predicting the frequency of cracking at various ages in the pavement life. The variables used involve stiffness of the original asphalt cement, winter design temperature, subgrade soil type, thickness of the asphalt concrete, and age of the pavement. Data for the variables are commonly recorded by highway agencies. The model provides a very powerful design approach in that serviceability losses and maintenance costs are directly related to degree of cracking. If these are intolerable, new designs can be generated and very quickly and easily tested by the model. The paper provides a numerical example, including a nomograph for solving the model. The model is "reasonable" in its use of the variables, and the degree of error involved is acceptable.

•NONTRAFFIC load associated cracking forms one of the three principal pavement design subsystems recognized by several authorities including The Asphalt Institute (1). In Canada and in the northern United States, this cracking of flexible pavements is primarily caused by low winter temperatures that induce tensile forces in the asphalt concrete. If the induced tensile forces exceed the tensile strength of the material, cracks are formed. Because the pavement cannot predominantly contract in the longitudinal direction, most low-temperature cracks are formed in the transverse direction to the highway route. These cracks are an annual multimillion dollar damage problem in Canada alone (2).

Various agencies and individuals have devoted considerable research effort to the problem. These investigations, both field- and laboratory-oriented, have led to the following design concepts for controlling or eliminating low-temperature cracking:

1. Limiting penetration and viscosity requirements on the asphalt cements;
2. Limiting strain or stiffness of the asphalt concrete (3, 4); and
3. Calculating fracture temperature [i. e., temperature at which the cracks are likely to be formed (5, 12)].

Although these concepts provide some very useful quantitative guidelines, they take into account only some of the variables that influence low-temperature cracking. Moreover, they do not allow predictions to be made of losses in pavement serviceability due to cracking during the pavement life.

The major objective of the study described in this paper was to develop a relation between field low-temperature pavement performance of asphalt pavements, in terms of low-temperature transverse cracking frequency, and variables of recognized significance on this frequency, which are usually recorded by highway agencies. Such a

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relation should make it possible to predict the low-temperature cracking frequency of newly planned highways at various ages. In turn, this will allow estimates to be made of the loss of pavement serviceability plus maintenance costs, due to cracking, in future years. If the estimates are considered to be excessive, a new design may be generated and again evaluated.

VARIABLES INFLUENCING LOW-TEMPERATURE CRACKING FREQUENCY

Variables influencing low-temperature cracking frequency and methods for their measurement have been presented elsewhere (6). In the following discussion, only factors of recognized significance in low-temperature cracking, which are generally recorded by highway departments and which will be subsequently used for the transverse pavement cracking prediction model, are considered.

Stiffness of Asphalt Cement or Mix

The influence of stiffness of the asphalt cement on low-temperature cracking has been reported and demonstrated by many investigators (3, 7). The major difficulty encountered here was the selection of the method to be used for obtaining asphalt and/or mix stiffness at temperatures when the cracks are likely to be formed. The methods for stiffness determination of bituminous materials at low temperatures may be classified as follows. Direct methods, based on direct testing of the materials; and indirect methods that estimate the stiffness modulus of bituminous materials by using their rheological properties at higher temperatures assessed by direct standard tests. Several various direct and indirect methods have been proposed and used.

A recent statistically designed experiment (8) showed that the efficient, indirect methods appear to estimate stiffness to a reasonably satisfactory degree necessary for prediction of low-temperature cracking frequency. As the best indirect method for the given purpose, a slightly modified version of McLeod's method was recommended. The selection of this method of stiffness determination has been made on the basis of 43 field observations (Tables 1 and 2) according to a statistical relation between stiffness moduli determined by various indirect methods and the appropriate low-temperature cracking frequency. The analysis also showed that the stiffness modulus of the original asphalt cement generally exhibits a greater degree of association with the cracking frequency than the stiffness moduli of field-aged asphalt cements and asphalt concretes (8). The use of stiffness of original asphalt cement has further advantages in that its value can be forecast on the basis of historical data.

Calculation of Stiffness

The stiffness of asphalt cement is calculated for a loading time of 20,000 sec and a temperature equal to a minimum ambient temperature chosen on a probabilistic basis of frequency and length of occurrence. Only the knowledge of penetration of the asphalt cement at 77 F and its viscosity at 275 F is required. From these two values, the penetration index (PI) may be determined, using Figure 1, based on the method recommended by McLeod (3). If the PI of the asphalt cement is known, we can use the nomograph shown in Figure 2 to calculate the value of the base temperature, which corresponds to the temperature of $T_{R\&D}$. In the final step, the stiffness modulus is estimated by using the nomograph shown in Figure 3.

Climatic Conditions

The results of Ste. Anne Test Road (7) demonstrated that most low-temperature transverse pavement cracks were initiated when the temperature decreased to a certain level for a certain time period. This shows that the formation of low-temperature cracks depends on overall winter climatic conditions. Thus, if we want to obtain a linear relation between cracking frequency and stiffness of asphalt cement (or mix), the ambient temperature that prevails most when cracks are initiated should be used for calculation of stiffness.

Because of the foregoing, a winter design temperature, defined as the lowest temperature at or below which only 1 percent of the hourly ambient air temperatures in January occur for the severest winter during a 10-year period, has been established in this study. This temperature was used to calculate stiffness moduli. The empirical relation between air freezing index (40-year average) and the winter design temperature developed on the basis of limited data for Ontario and southern Manitoba is shown in Figure 4. This relation can be modified as more data are available. In the meantime, it can serve as a guide for estimating winter design temperatures when only freezing indexes are known.

Age of Pavement

Results of transverse crack frequency surveys, involving more than 1,900 miles of paved highways in Alberta (10), showed an increase of cracking frequency with pavement age. The increase of low-temperature cracking frequency can be caused both by increase of stiffness of asphalt mix and by increase of the probability of occurrence of more extreme low temperatures, which increases with pavement age.

Thickness of Asphalt Concrete Layer

The Ste. Anne Test Road (7) showed that increase of the thickness of asphalt concrete layer (from 4 to 10 in.) results in only half of the low-temperature cracking frequency when all other variables were the same. This difference in pavement performance may be explained by the relatively good insulating properties of asphalt concrete (8).

Pavement Foundations

It has been reported several times that the transverse cracking frequency for pavements placed over sand subgrades is considerably higher than for pavements placed over clay subgrades, all other variables being constant (6, 7). A similar effect on pavement performance caused by a 16-in. thick granular base was reported by McLeod (11).

MATHEMATICAL MODEL

Purpose of Mathematical Model

The foregoing discussion showed that low-temperature cracking frequency depends on many variables, some of which interact with one another. A mathematical model enables one to produce an overall functional relation when one variable, in this case cracking frequency, is a multivariable function of a large number of independent variables, i.e., stiffness of asphalt concrete, age and thickness of asphalt concrete layers, winter design temperature, and type of subgrade.

Description of Data Used

From the practical point of view, the aim of the mathematical model is to predict the low-temperature cracking frequency of future asphalt concrete pavements on the basis of past experience, that is, to make generalizations from observed data in space and time. To increase the scope of generalization as well as to provide unbiased estimates within this scope, one should choose the observations used for construction of a mathematical model in a random manner from all possible observations in the space where the future predictions are to be made. Because this requirement could not be satisfied, we decided to use all available observations that satisfied given requirements to eliminate a personal bias. It was assumed that the observations that would become available during the course of the work, or later on, could be used for checking purposes.

The observations used in this study, most of which were obtained through the courtesy of the Department of Transportation and Communications of Ontario (DTCO), are summarized in Table 1. The DTCO data were collected from subobservations by using an arithmetic mean. The observations based on the results of the Ste. Anne Test Road were extracted in a similar manner from tables given elsewhere (7).

Table 1. Transverse pavement cracking data.

Observation Number (1)	Source (2)	Locality and/or DTCO Contract Number (3)	Transverse Cracks per 500 ft: W (4)	X (5)	Y (6)	Z (7)	CI (8)	Original Asphalt: Pen. at 77 F (9)	Original Asphalt: Vis. at 275 F (cs) (10)	PI (11)	AC Stiffness (kg/cm²) (12)	FI (degree-days) (13)	WDT (deg C) (14)	AC Thickness (in.) (15)
1	D	Arkona A	1	10	21	36	21	98	151	-1.8	80	650	-20	3.7
2	D	Arkona B	0	0	1	9	1	91	368	-0.6	25	650	-20	3.7
3	D	Arkona C	0	1	0	2	1	95	427	-0.3	22	650	-20	3.9
4	D	63088, Grand Bend	0	0	0	0	0	87	338	-0.7	31	650	-20	2.2
5	D	61078, Highway 11	0	7	3	10	9	84	425	-0.3	301	1,850	-30	4.6
6	D	61025, South River	2	17	5	10	22	87	208	-1.4	630	1,880	-30	4.2
7	D	60102, Highway 11	0	10	10	54	15	171	141	-1.3	170	1,900	-30	4.0
8	D	Sault Ste. Marie to Heyden	0	3	3	5	5	89	331	-0.7	230	1,650	-28	5.5
9	D	Heyden to Haviland Bay	2	10	18	18	20	96	211	-1.3	400	1,750	-28	3.2
10	D	Agawa	0	1	0	0	1	161	256	-0.3	220	2,650	-35	4.5
11	D	60041, Highway 17	0	1	0	1	1	145	222	-0.7	370	2,650	-35	3.1
12	D	Orangeville A	0	10	37	61	23	84	205	-1.4	320	1,450	-26	3.2
13	D	Orangeville B	0	1	10	6	6	157	142	-1.3	66	1,450	-26	3.1
14	D	58277, Highway 17	0	2	1	4	3	175	270	-0.8	60	1,650	-28	3.4
15	D	Val Albert easterly	0	10	43	22	33	153	226	-0.6	900	3,450	-40	2.1
16	D	Val Albert	0	1	2	7	2	153	223	-0.7	1,000	3,450	-40	2.0
17	D	62065, Madoc-Ivanhoe	0	0	0	0	0	85	420	-0.3	125	1,400	-26	2.3
18	D	56384, Highway 28	0	1	1	0	2	176	252	-0.2	61	1,550	-27	5.3
19	D	62185, Highway 62	0	0	0	0	1	151	250	-0.5	51	1,550	-27	2.5
20	D	61124, Highway 69	0	6	17	19	15	87	382	-0.5	150	1,480	-26	4.6
21	D	61113, Parry Sound	0	1	13	27	8	85	450	-0.3	125	1,480	-26	3.0
22	D	62609, Highway 12 Orillia	0	6	5	13	8	83	216	-1.4	360	1,500	-26	5.0
23	D	60183, Highway 62	0	4	1	1	5	168	226	-0.5	80	1,900	-30	3.7
24	D	61052, Highway 62	0	2	0	0	2	168	243	-0.3	85	1,880	-30	3.7
25	D	60131, Highway 41 Caledon	0	0	0	3	0	154	203	-0.8	60	1,500	-27	3.3
26	7	Ste. Anne Road Test	-	62	0	-	63	192	110	-1.6	1,850	3,250	-40	4.0
27	7	Ste. Anne Road Test	-	-	-	-	0	159	225	-0.5	680	3,250	-40	4.0
28	7	Ste. Anne Road Test	-	-	-	-	0	313	86	-1.4	540	3,250	-40	4.0
29	7	Ste. Anne Road Test	0	25	0	0	25	192	110	-1.6	1,850	3,250	-40	4.0
30	7	Ste. Anne Road Test	-	-	-	-	0	159	225	-0.5	680	3,250	-40	4.0
31	7	Ste. Anne Road Test	0	8	0	0	8	313	86	-1.4	540	3,250	-40	4.0
32	7	Ste. Anne Road Test	0	10	1	0	10	192	110	-1.6	1,850	3,250	-40	9.9

Note: Original asphalt properties of observations 1 and 2 are those given by Gulf Oil Canada Ltd. In column 2, D = DTCO, and 7 is reference number. In columns, 4, 5, 6, and 7, W = multiple, X = full, Y = half, and Z = partial. In column 8, CI = cracking index. In column 9, penetration of 100 grams for 5 sec. In column 13, freezing index is based on a 40-year average. Winter design temperatures given in column 14 are derived from column 13 according to Figure 4. In column 19, 12 and 7 are reference numbers, and P = estimated by using a pedologic map.

Table 2. Check on model.

Observation Number	Locality and/or DTCO Contract Number	Original Asphalt: Pen. at 77 F	Original Asphalt: Vis. at 275 F (cs)	Climate: FI (degree-days)	Climate: WDT (deg C)	AC Stiffness (kg/cm²)	AC Thickness (in.)	Age (years)	Subgrade: Description	Subgrade: Method
33	60181, Highway 17, Black River	183	171	2,650	-35	270	1.9	6	Lacustrine deposits	G[a]
34	59177, Highway 11, New Liskeard	170	139	2,800	-36	720	5.0	7	Silty clay loam	P[b]
35	60138, Highway 11, Earlton	144	277	2,830	-36	350	3.1	5	Clay	P
36	62030, Highway 11, Englehart	180	313	2,850	-36	120	3.1	3	Mostly silt and loam	P
37	53023	90	350	1,450	-26	155	4.1	13	Unknown	-
38	57121, Highway 17, Iron Bridge	180	306	1,650	-28	25	3.1	9	Varved or massive clay and silt	G
39	52080	90	350	1,150	-24	115	3.9	14	Unknown	-
40	59038, Highway 17, North Bay	164	259	2,000	-30	80	3.6	8	Mostly sandy till	G
41	Brampton, test road, sections 16, 18, 35, 36	89	312	700	-20	25	5.5	6	Clay	D[c]
42	60122, Highway 17, North Bay	159	139	2,000	-30	215	3.0	7	Mostly sandy till	G
43	Saskatoon Airport, Taxi A	158	315	4,000	-42	420	6.0	9	Silty clay	T[d]

Note: For an explanation of abbreviations, see Table 1.

[a]G = map of surficial geology. [b]P = pedologic map. [c]D = DTCO. [d]T = Canadian Department of Transport.

Age (years) (16)	Subgrade Description (17)	Code (18)	Method (19)	No. Sub-observations (20)	Observation Number (1)
6	Clay	2	12	10	1
6	Clay	2	12	9	2
6	Clay	2	12	7	3
3	Sand	5	P	2	4
5	Sandy loam	3	P	8	5
4	Sandy loam	3	P	6	6
6	Sandy loam	3	P	2	7
5	Clean granular	5	12	6	8
6	Clean granular	5	12	8	9
6	Sandy loam	3	12	3	10
6	-	-	-	3	11
6	Silty sand	3	12	3	12
6	Silty sand	3	12	3	13
8	Sandy loam	3	P	3	14
3	Deep sand fill	5	12	5	15
3	Clayey sand	2	12	7	16
5	Clayey loam	2	P	3	17
11	Sandy loam	3	P	3	18
5	Gravelly sandy loam	3	P	2	19
5	Sandy loam	3	P	6	20
6	Sandy loam	3	P	4	21
5	Sandy loam	3	P	6	22
7	Sandy loam	3	P	6	23
6	Sandy loam	3	P	4	24
7	Sandy loam, rocky phase	3	P	3	25
2	Sand	5	7	-	26
2	Sand	5	7	-	27
2	Sand	5	7	-	28
2	Clay	2	7	-	29
2	Clay	2	7	-	30
2	Clay	2	7	-	31
2	Clay	2	7	-	32

Cracking Index: CI for Subobservations	Estimated Sand	Estimated Loam	Estimated Clay	Agreement	Observation Number
11, 4, 4	6.2	6.9	6.1	Yes	33
16, 40	-	21.6	16.1	Yes	34
4, 6, 25, 48	-	-	4.5	?	35
5, 1, 2, 1	0	0	0	Yes	36
14, 13	18.3	20.5	21.0	Yes	37
2, 7, 2, 2, 1, 9	-	0	0	Yes	38
0, 4, 6, 12, 17, 20	18.0	20.7	21.7	?	39
0, 1, 0, 0, 2, 2	0	1.7	-	Yes	40
0	-	-	2.5	Yes	41
17, 18, 24	9.3	10.7	-	No	42
0, 1	-	-	2	Yes	43

Figure 1. Modification of McLeod's graph for estimation of penetration index.

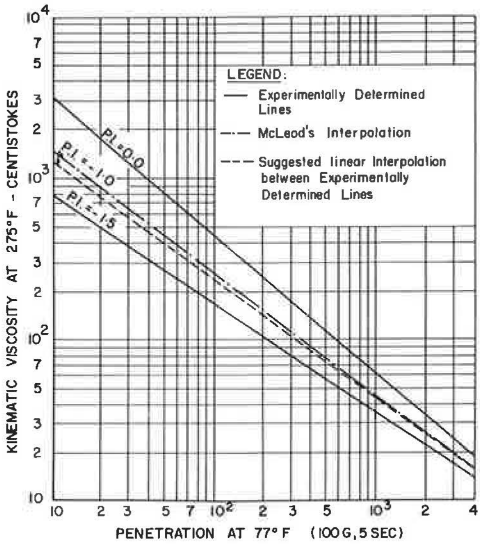


Figure 2. Pfeiffer's and Van Doormaal's nomograph for determination of penetration index.

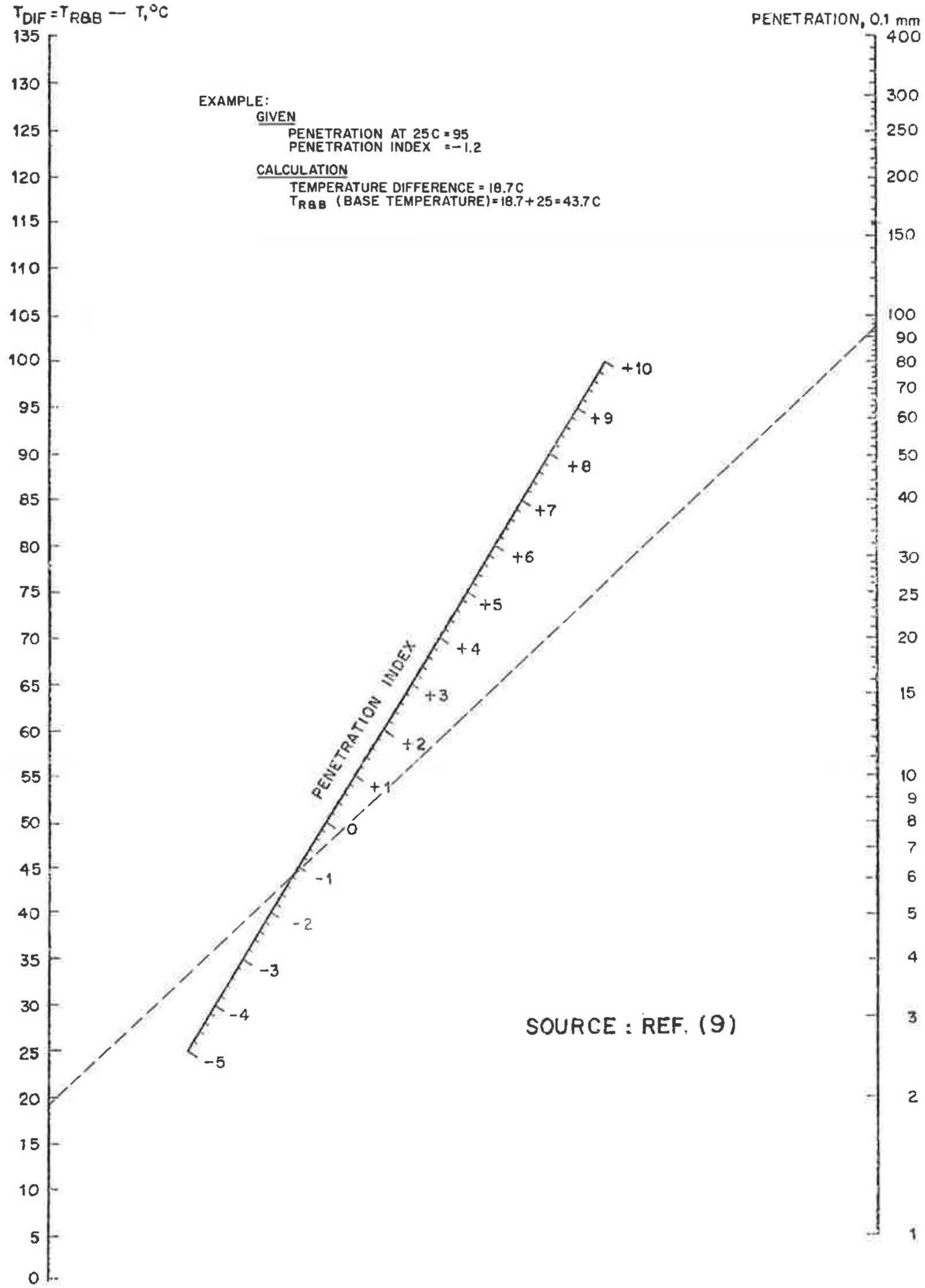
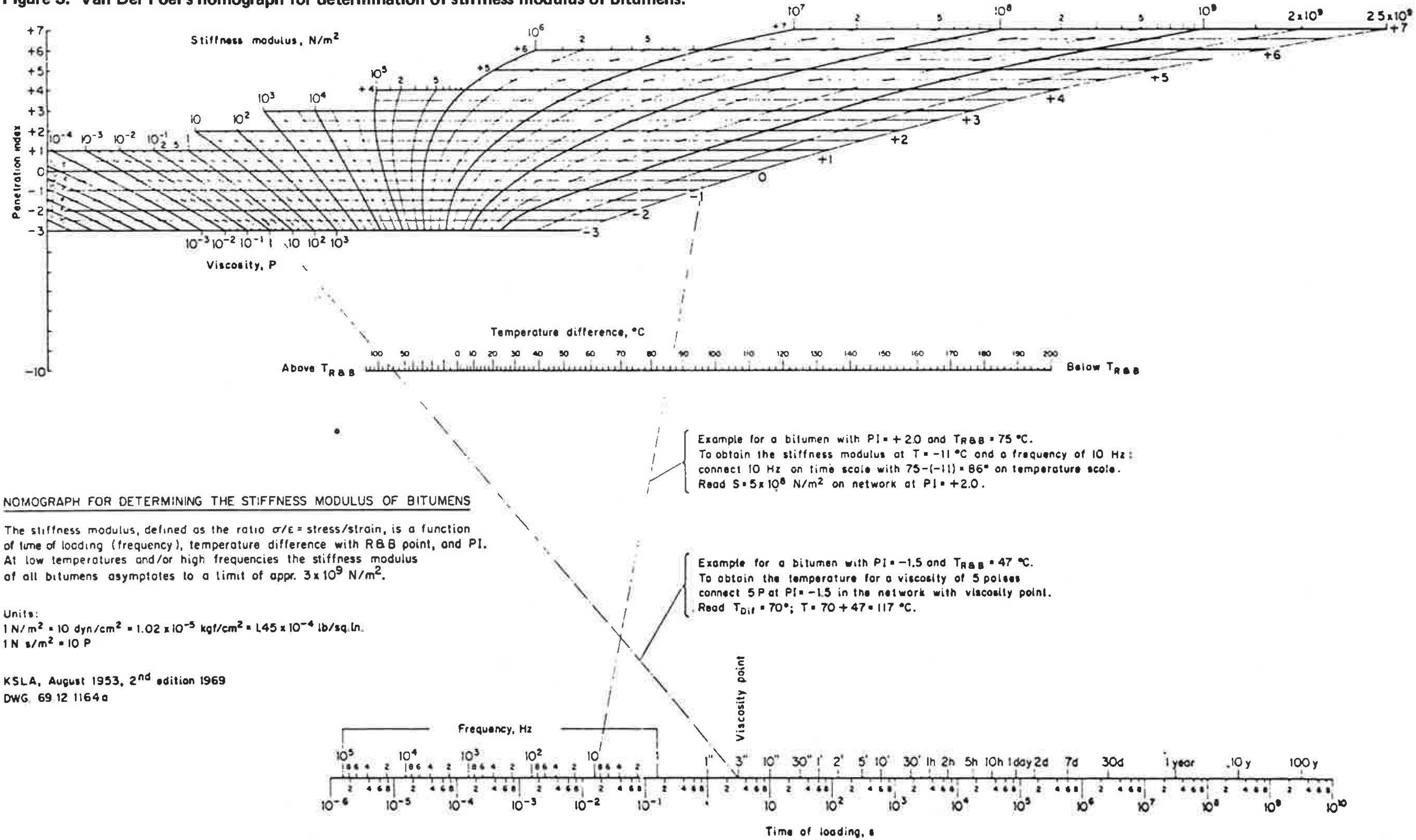


Figure 3. Van Der Poel's nomograph for determination of stiffness modulus of bitumens.



Dependent Variable—Definition of Cracking Index

The cracking index, introduced by the DTCO and defined as the number of full and half transverse cracks per 500-ft section of two-lane highway, does not include transverse cracks less than one-half of roadway width. The assumption is that such cracks usually occur subsequent to the formation of half and full cracks, and therefore they are not a primary manifestation of low-temperature pavement cracking (12). It should be pointed out that the values of cracking indexes given in Table 1 are based on field counts. Thus, they refer to transverse pavement cracking frequency (13) rather than to the low-temperature cracking frequency per se.

Selection of Independent Variables

The stiffness of original asphalt cement, determined by the modified McLeod method, was selected as the sole representative of asphalt and/or mixture properties related to low-temperature cracking frequency. In addition to the reasons behind this decision given previously, the use of the stiffness modulus has the advantage of combining several variables into one and thus reducing the complexity of the mathematical model. It was felt that the addition of variables not included in the model (i. e., aggregate absorption, coefficient of thermal expansion of aggregate, and thickness of granular material forming the subbase) might result in improvement of both statistical validity and applicability of the model, but values of these variables were not available.

Construction of Mathematical Model

Construction of the model consisted of finding a function that relates the cracking index, I, to other variables, that is, to find

$$I = f(s,\ t,\ a,\ m,\ d)$$

where

- I = cracking index, $I \geq 0$;
- s = stiffness of original asphalt cement determined for temperature m and loading time of 20,000 sec by modified McLeod method, $(\mathrm{kg/cm^2}) \times 10^{-1}$;
- t = thickness of all asphalt concrete layers, in.;
- a = age of asphalt concrete layers, years;
- d = type of subgrade (dimensionless code: 2 is clay, 3 is loam, and 5 is sand); and
- m = winter design temperature, deg C × -0.10.

The determination of the function could not be deduced solely on the basis of theoretical mathematical considerations and past experience. Only the following limiting conditions could be set forth:

1. Whenever the modulus of stiffness equals zero, the function must give zero as a value. The same requirement may be set for age, but it can be assumed that, whenever this variable is equal to zero, the modulus of stiffness is inevitably equal to zero as well because of the interaction of variables.
2. With increase of stiffness and/or age, the cracking index should increase when all other variables are fixed.
3. With unlimited increase of thickness, the cracking index should decrease without limit if all other variables remain constant.

The following suggested model was arrived at by testing and evaluating more than 20 various functions:

$$10^{I} = c_1 \times s^{(c_2 + c_3 t + c_4 a)} \times c_5^{d} \times c_6^{m} \times d^{c_7 s} \tag{1}$$

where I, s, t, a, m, and d are the original dependent and independent variables defined previously; and c_1, c_2, ..., c_7 are constants of the model.

Estimation of constants in the model has been done in two steps:

1. The model represented by Eq. 1 was linearized by taking logarithms of both sides of the equation as follows:

$$I = \log c_1 + c_2 \log s + c_3 t \log s + c_4 a \log s + \log c_5 d + \log c_6 m + c_7 s \log d \quad (2)$$

where $\log c_1$, c_2, c_3, ..., c_7 are partial regression coefficients, and $\log s$, $t \log s$, $a \log s$, ..., $\log d$ are transformed independent variables.

2. The constants of the linearized model (regression equation) were calculated by using the least squares method and a stepwise regression computer program. The sequence of transformed independent variables added to the regression equation, and their contribution to the increase of the proportion of the total variance explained by the regression equation is given in Table 3. The abbreviated printout for the last step is interpreted in Tables 4 and 5.

By substituting the calculated constants given in Tables 4 and 5 into Eq. 1 and taking appropriate antilogs where necessary, we get the suggested mathematical model for prediction of low-temperature cracking frequency of asphalt concrete pavements in the following form:

$$10^{I} = 2.497 \times 10^{30} \times s^{(6.7966 - 0.8740\,t + 1.3388\,a)} \times (7.054 \times 10^{-3})^{d} \times (3.193 \times 10^{-13})^{m} \times d^{0.6026\,s} \quad (3)$$

Equation 3 shows that the model satisfies the limiting conditions, provided that stiffness of asphalt cement is equal to or greater than 1.0 kg/cm^2.

EVALUATION OF THE MODEL

The model was evaluated in terms of its statistical significance, rational behavior, and relation to observed data. Also, limitations of the model were outlined.

Statistical Evaluation

The two biggest values of the simple regression coefficients given in Table 6, $r_{5,6}$ and $r_{6,7}$, suggest that there is a degree of association between the pairs of transformed independent variables 5,6 and 6,7. Because the dependencies between these pairs of independent variables are nonlinear, they can be included in the regression equation (14).

The significance of the multiple correlation coefficient of the model, R = 0.91, can be judged by the method developed by Fisher (15). We can say, according to the method, that the true correlation is at least 0.81 in the universe from which the sample was drawn, with 1 change in 20 being wrong on the average. Table 3 shows that 82.2 percent of the total variance is explained by the regression equation. The standard error of estimate is 6.2 of the cracking index. By comparing this standard error of estimate with the average observed standard deviation of the cracking index (calculated for DTCO observations aggregated from more than two subobservations), which is 3.61, we can conclude that the standard error of estimate of 6.2 of the cracking index is comparable. For example, the observed standard deviation of the cracking index for the Arkona Test Road, section A, is 4.4. Statistical significance of the partial regression coefficients of the transformed independent variables is given in Table 5.

Rational Behavior of the Model

The behavior of the model and the isolated effects of the independent variables may be best demonstrated by means of a graphic presentation. The model has five original independent variables. If values are assigned to any four of these, the resulting equation may be plotted in the form of a two-dimensional graph (dependent variable versus original independent variable), which represents a 2-variable trace of the multidimensional space defined by the model.

Figure 4. Relation between freezing index and winter design temperature.

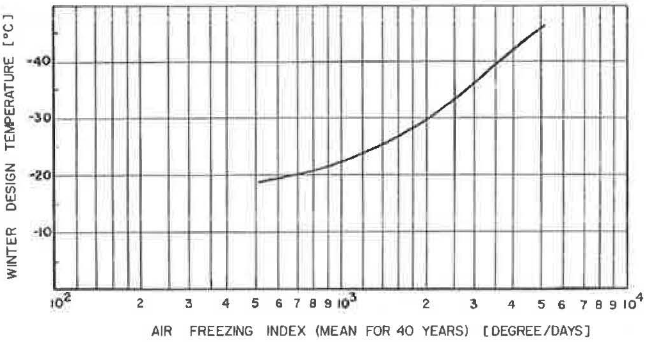


Table 3. Summary of stepwise regression analysis.

Step Number (1)	Transformed Independent Variable Included (2)	Multiple Correlation Coefficient R (3)	R^2 (4)	Increase in R^2 (5)	F-Value to Enter the Equation (6)	Significance of Addition (percent) (7)	Standard Error of Estimate (cracking index) (8)
1	s log d	0.7570	0.5730	0.5730	40.2562	0.5	8.8228
2	m	0.8491	0.7210	0.1480	15.3814	0.5	7.2538
3	a log s	0.8787	0.7721	0.0511	6.2750	2.5	6.6723
4	d	0.8906	0.7931	0.0210	2.7469	20	6.4734
5	t log s	0.9021	0.8138	0.0207	2.8895	20	6.2582
6	log s	0.9068	0.8222	0.0084	1.1868	30	6.2358

Note: In column 1, step number equals the number of transformed independent variables included in regression equation. In column 4, R^2 multiplied by 100 gives the percentage of the total variance explained by the regression equation in a given step. In columns 6 and 7, 6.2750, for example, means that by adding variable a log s to the previous two variables already in the equation, statistical effect of this addition measured by F-test is equal to 6.2750. For the appropriate degrees of freedom (n_1 = 1 and n_2 = 28), 6.2750 is significant at 2.5 percent probability level (column 7). We can conclude that the addition of the third variable a log s results in a significant improvement of the equation with only two independent variables having 2.5 percent probability of being wrong.

Table 4. Analysis of variance.

Variance	Degrees of Freedom	Sum of Squares	Mean Square	F Ratio	Level of Significance (percent)
Due to regression	6	4,496.723	749.454	19.273	0.1
Residual	25	972.130	38.885	-	-
Total	31	5,468.853	176.414	-	-

Note: Multiple correlation coefficient = 0.9068. Standard error of estimate = 6.2358.

Table 5. Partial regression coefficients.

Transformed Independent Variable	Partial Regression Coefficient	Standard Error	Value of Student t Test	Level of Significance[a] (percent)
s log d	0.6026	0.0836	7.18	0.1
m	-12.4958	3.3902	3.68	0.5
a log s	1.3388	0.6570	2.03	10.0
d	-2.1316	1.1808	1.80	10.0
t log s	-0.8740	0.4340	2.02	10.0
log s	6.7966	6.2387	1.09	25.0

Note: Constant term of regression equation = 30.3974.

[a]Refers to the 2-sided test for zero hypothesis.

Table 6. Correlation matrix of linearized model.

Variable	i	t log s	a log s	d	m	log s	s log d
i	1.000	0.265	0.150	0.233	0.240	0.502	0.757
t log s		1.000	0.059	-0.012	0.580	0.726	0.560
a log s			1.000	0.128	-0.012	0.192	-0.072
d				1.000	0.172	0.156	0.374
m					1.000	0.852	0.687
log s						1.000	0.782
s log d							1.000

This has been done in Figures 5 through 9 for observations 1, 2, 6, and 29 given in Table 1. The assumption is that the original independent variable, for which the 2-variable trace is plotted, can obtain any value (in the range used for the construction of the model), whereas values of all remaining original independent variables, for the given observation, are fixed.

Thickness of Asphalt Concrete Layers

The isolated effect of thickness of asphalt concrete layer on the cracking index is shown in Figure 5. The increase of thickness of asphalt concrete layer results in a decrease of low-temperature cracking frequency. The decrease is not constant for different highway sections. This is quite rational because we cannot expect a low cracking index, observed after several years of pavement service, to be decreased by the assumed increase of thickness to the same degree as a considerably high cracking index, particularly if the latter one was observed for an area with lower winter design temperature.

Age of Asphalt Concrete Layers

The separated effect of age of asphalt concrete pavements on the cracking index (Fig. 6) is in many respects similar to the effect of thickness. In this case, however, the index increases with an increased pavement age. Again, we cannot expect that the same increase of cracking index will result for different highway sections.

Type of Subgrade

Figure 7 shows the isolated effect of type of subgrade, identified by the given code, on the cracking index. With a change of subgrade from clay to sand, the cracking index for the observations with extremely low winter design temperatures (i. e., -40 C) significantly increases. For observations of (-20 C) winter design temperatures, type of subgrade does not exhibit a significant effect. This behavior of the model may be considered rational on the basis of the following explanation.

The initial decrease of temperature below the freezing point results in formation of ice crystals, which may be accompanied by an increase in the volume of soil. This increase is probably more pronounced in the case of sandy soils because practically all their water becomes frozen. At the same time, the decrease of temperature causes contraction of the soil particles. According to the model, the effect of the initial increase of volume of sandy soils, caused by formation of ice crystals, is overcome by shrinkage induced by further temperature decrease at a winter design temperature of approximately -30 C. The quite different effect of clay and sand subgrade on the cracking index at this and lower temperatures was explained by Hajek (8), who used a soil-water retention theory.

Stiffness of Original Asphalt Cement

The isolated effect of the stiffness modulus of original asphalt cement on the cracking index is shown in Figure 8. As stiffness increases, the cracking index increases.

Winter Design Temperature

Figure 9 shows that, when winter design temperature decreases, the cracking index decreases as well. This is quite rational because the winter design temperature was used for calculation of stiffness. For example, in the case of observation 29, the model predicts a cracking index of 23.3 for a winter design temperature that is equal to -40 C. The observed cracking index for this condition was 25. If the four other original variables of the model (thickness, age, stiffness, and type of subgrade) are held constant and the winter design temperature is assumed to be equal to -30 C, the predicted cracking index equals 35.8. This is logical because stiffness calculated for the temperature of -40 C is assumed to be the same as stiffness calculated for the temperature of -20 C.

Figure 5. Predicted effect of thickness on cracking index.

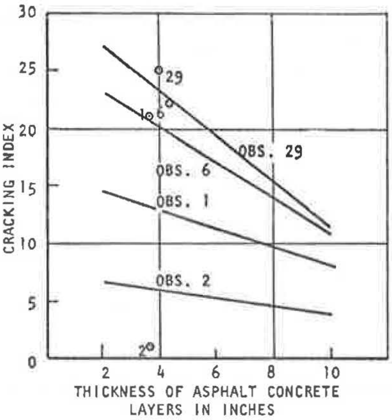


Figure 6. Predicted effect of age on cracking index.

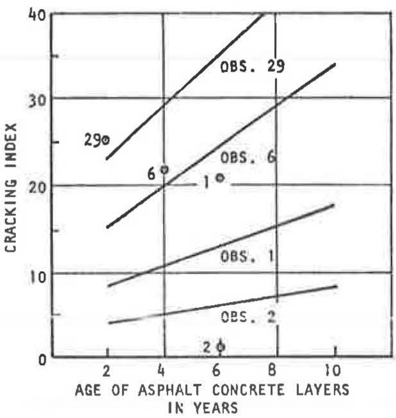


Figure 7. Predicted effect of type of subgrade on cracking index.

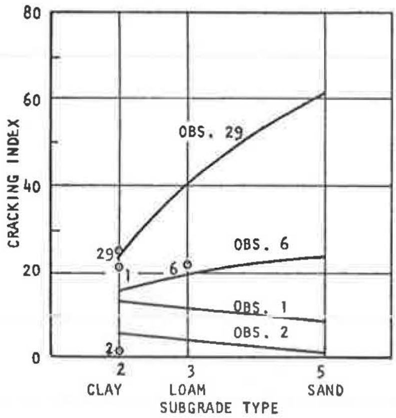


Figure 8. Predicted effect of stiffness modulus on cracking index.

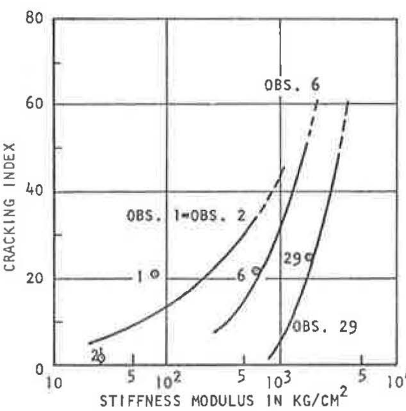
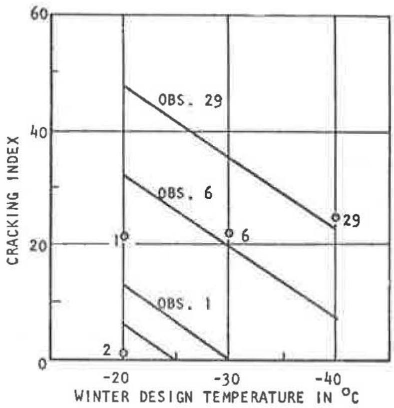


Figure 9. Predicted effect of winter design temperature on cracking index.



Relation to Observed Data

The observed values of the cracking index are plotted against values estimated by the model shown in Figure 10. Generally, a good agreement may be noted; nevertheless, for several observations a considerable difference between estimated and observed values of the cracking index was obtained. Several explanations of this fact can be offered, and some of them are discussed in the section devoted to limitations of the model.

Testing of the Model

The model was tested using all additional observations that had become available. Some of these observations were not used for construction of the model because of the uncertainty encountered in determining subgrade type. The result of the check, given in Table 2, suggests that the model can be used for design purposes with a reasonably high degree of confidence.

Limitations of the Model

The most significant limitation, imposed by the regression analysis used for the calculation of constants and the statistical evaluation of the model, was the assumption that all independent variables are measurable on interval or ratio scales, which was not the case for the variable type of subgrade. On the other hand, Hajek (8) in an extensive discussion showed that the use of a different code for expressing the effect of subgrade type (i. e., 1 is sand, 2 is loam, and 3 is clay) would result in a very similar prediction model as far as its statistical significance and rational behavior are concerned. The present difficulties encountered in developing the interval scale for measuring the effect of type of subgrade on low-temperature cracking frequency may be overcome by developing separate prediction equations for different subgrade types—on the condition that enough observations are available (8).

The additional limitations inherent in the model are as follows:

1. The suggested model may not represent the "true law" that governs low-temperature transverse cracking frequency. Moveover, the 32 observations used for construction of the model are not usable (so far as their number and selection are concerned) as a basis for establishing such a "true law."
2. Extrapolation outside the range of values of the original variables used is "risky."
3. The least squares solution provided by the regression analysis is valid for the linearized model (Eq. 2) but not for the original model (Eqs. 1 and 2).
4. The model was constructed to fit the cracking index, but the model predicts the low-temperature transverse cracking frequency, assuming as was demonstrated (7) that most of the transverse pavement cracks in the region investigated are low-temperature cracks. The model does not have, for example, any provision for including transverse cracks caused by shrinkage associated with absorptive aggregates (16), reflection transverse pavement cracks through overlays, and so forth.

Example Application

The major significance of the suggested model is seen in its immediate capacity to provide an engineering solution to the problem of minimizing low-temperature transverse pavement cracking in North America. For an illustration of the procedure involved, the following numerical example is provided.

Let us assume that we want to predict the low-temperature transverse cracking frequency of a newly planned asphalt concrete pavement throughout its service life. The following variables are known:

1. The ambient freezing index (40-year average) is estimated as equal to 1,650 degree-days;
2. Thickness of asphalt concrete pavement is 4 in.;
3. Reference to pedological and topographical maps reveals that we can expect poorly graded sands with some fines as a subgrade type; and

Figure 10. Observed cracking index versus cracking index estimated by model.

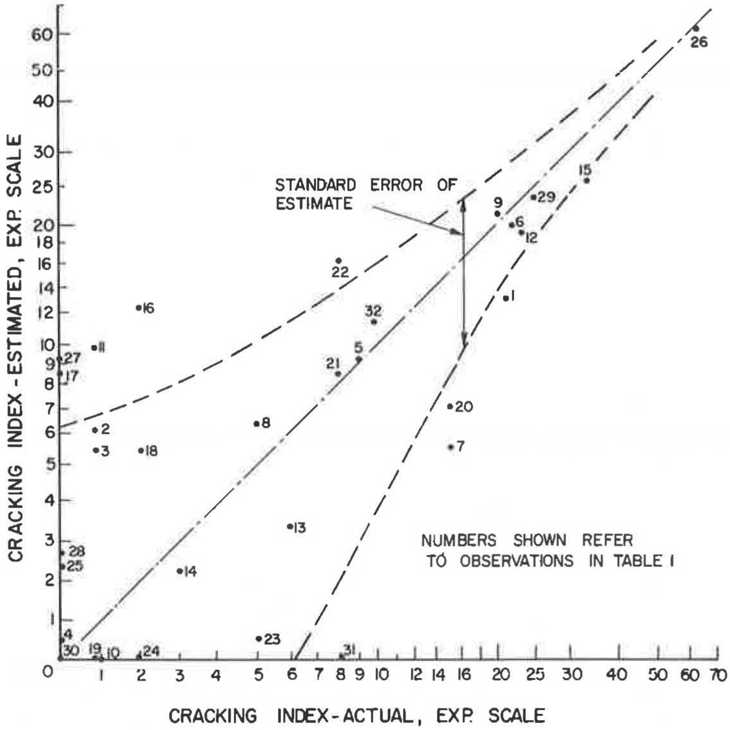
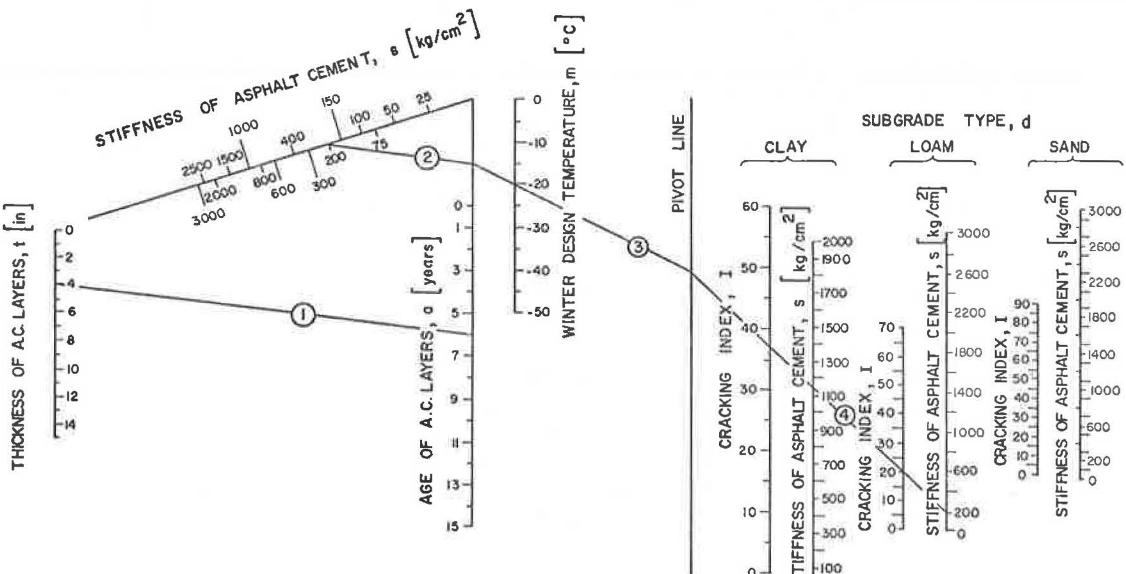


Figure 11. Nomograph for predicting low-temperature cracking frequency of asphalt pavements.

MODEL : $10^{I} = 2.4970 \times 10^{30} \cdot s^{(6.79660 - 0.87403t + 1.33884a)} \cdot (7.0539 \times 10^{-3})^{d} \cdot (3.1928 \times 10^{-13})^{m} \cdot d^{0.60263s}$



EXAMPLE:

thickness = 4 inches
age = 6 years
stiffness of original asphalt cement = 200 kg/cm^{2} *
winter design temperature = -20°C
subgrade type = loam
*for temp. = m and time = 20000 sec.

NOTE:

A) lines (1) and (2) are parallels
B) in step (4) select scales for the appropriate subgrade type

RESULT: cracking index = 20 at 6 years

4. The asphalt cement to be employed has the following original properties—penetration at 77 F, 100 gram, 5 sec = 95 and viscosity at 275 F = 210 cs.

The computation procedure consists of the following steps:

1. The expected minimum ambient temperature, equal to -27 C, is derived from the freezing index (Fig. 4);
2. Penetration index of asphalt cement, equal to -1.2, is obtained by means of Figure 1;
3. Ring and ball temperature, equal to 43.7 C, is determined by using the nomograph shown in Figure 2;
4. Modulus of stiffness of original asphalt cement is estimated by using the nomograph shown in Figure 3. [For a temperature -27 C (see step 1) and for a loading time of 20,000 sec, stiffness modulus of asphalt cement is equal to 205 kg/cm^2];
5. On the basis of the ordinal scale described (5 is sand, 3 is loam, and 2 is clay), it was decided to use code 5 for the type of subgrade; and
6. By substituting all required variables into Eq. 3 and solving for the cracking index (or by using the nomograph shown in Fig. 11), we get a cracking index of 7.7 for an age of 5 years and 16.5 for 10 years.

The standard error of the estimate is 6.2. Finally, if the designer concludes that the loss of pavement serviceability due to cracking may be too high for the particular conditions (subgrade type and thickness of asphalt concrete layer), he may use an asphalt cement with a lower modulus of stiffness by using softer asphalt cement and/or less temperature-susceptible asphalt cement (e. g., a 150- to 200-penetration asphalt of the same source). If all other variables remain equal, then a significantly lower cracking index would result. If such a change does not violate any other constraints (i. e., mix stability, fatigue life requirements, and so forth), the softer asphalt cement should be employed as a new design.

Additional Comments on the Modeling Approach Used

This paper has demonstrated only that a quite reasonable design model for predicting low-temperature cracking frequency of asphalt pavements can be developed from easily obtainable data. It must be emphasized though that this is not necessarily the best or the only model possible. Certainly, more data on additional variables and from more sections would be desirable.

It must also be emphasized that, because the model uses indirectly determined stiffness values, this does not necessarily imply that stiffness determination is most reliable from such methods. The available data only permitted the indirect type of analysis. For control or specification purposes, the use of direct, fundamental methods of stiffness determination has been recommended (1).

CONCLUSIONS

Low-temperature transverse cracking of asphalt pavements is a serious and costly problem in many parts of North America. Considerable effort has been devoted to investigating the problem and finding solutions.

This paper demonstrates that it is now possible to develop a model for predicting the frequency of such cracking. A very powerful design tool is thereby provided in that serviceability losses and maintenance costs can be related to degree of cracking.

The model presented in this paper is considered to be "reasonable" and to have an acceptable limit to the error involved. Moreover, only easily obtainable data are required, and the use of the model in design is very simple and quick.

ACKNOWLEDGMENTS

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